of concrete on steel embedded in it was not widely disseminated, and Mr. Babcock decided to protect the steel diaphragm with an asphalt and burlap covering.

As the diaphragm was built up, it was mopped with hot asphaltum and a layer of burlap was placed against

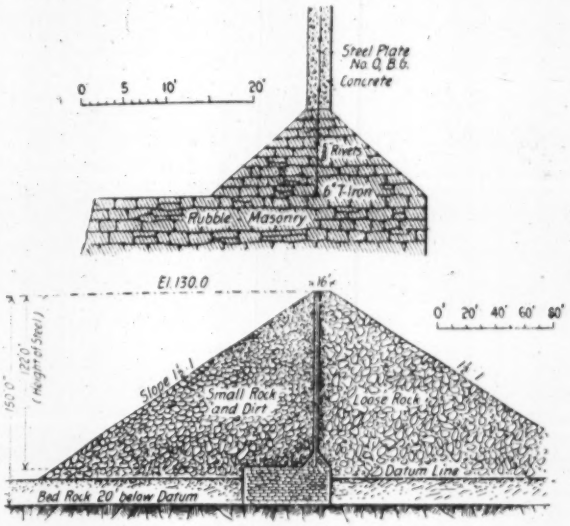


FIG. 2. CROSS-SECTION OF LOWER OTAY DAM AND ENLARGED SECTION OF STEEL CORE WALL AT BASE

Reproduced from "Engineering News," Mar. 10, 1898; dimensions of dam as built vary from this

it and held in place by boards. Then a thin wall of cement masonry, about a foot thick, was carried up on each side of these boards, being held between wooden forms which were left in place. The rock was deposited directly against the concrete as the dam was built up. The purpose of the concrete was rather to protect the steel from mechanical injury than to keep water away from the steel.

According to information furnished the writer by those directly connected with the work of construction, slap-dash methods were pursued. The rock was excavated by huge blasts in the cañon walls just below the dam and was deposited in the dam by cableway. The steel-plate diaphragm was built up often considerably in advance of the concrete covering. Being exposed to the intensely hot sun of southern California, the top of the diaphragm expanded and assumed a series of curves, so that it was impossible to do a good job in holding the layers of asphalt, burlap, boards and concrete against the steel. There was of course no way in which to determine whether the riveting and calking of the diaphragm was a water-tight job.

In depositing the rock the finer material was placed on the upstream side and the large pieces on the downstream side, but this was merely to secure as much strength as possible on the downstream side. It is not surprising under the conditions above recited that the dam was not water-tight. It has always leaked. It has been claimed that the leakage is through seams in the porphyry rock in which the dam is founded, but there was no means of ascertaining how much of the water might be coming through the steel diaphragm.

Leaky Dam Doubly Serious at San Diego

Leakage through a dam is always objectionable, but on the San Diego water-supply it is a very serious matter. The average annual rainfall at San Diego is only 9 in. per annum and only some 13 in. on the Otay drainage basin. The annual rainfall, however, is subject to great variation, as was shown by the curves accompanying the paper by Allen Hazen, on variation in annual rainfall, published in *Engineering News* of Jan. 6, 1916. During three successive years, from 1901 to 1904, the annual rainfall at the Sweetwater dam, a half dozen miles away on the next stream north of Otay, averaged less than 5½ in., and there was no runoff whatever into the reservoir. That is why leakage of the water stored for San Diego's use is a serious matter.

According to the last annual report of the San Diego water department, the daily leakage from the Lower Otay dam was 225,000 gal.; but the writer was told at the time of his visit last October that the leakage was as much as 400,000 gal. a day. In order to save this water a sump and a pumping plant operated by a 50-hp. oil engine were built below the dam in 1913 to pump back into the reservoir the water that leaked through the dam.

It will be at once recognized that the entire strength of the dam to resist the water pressure against it lies in its lower half below the steel diaphragm, for the porous fill on the upper side admits the water freely. Further, this lower half of the dam has to act as a retaining wall to sustain the pressure of the upper half of the dam in addition to the water pressure.

The accompanying cuts, reproduced from the article published in *Engineering News* of March 10, 1898, show a structure with slopes of 1½ horizontal to 1 vertical, but the slopes of the dam as actually built are steeper than this. The writer was distinctly impressed

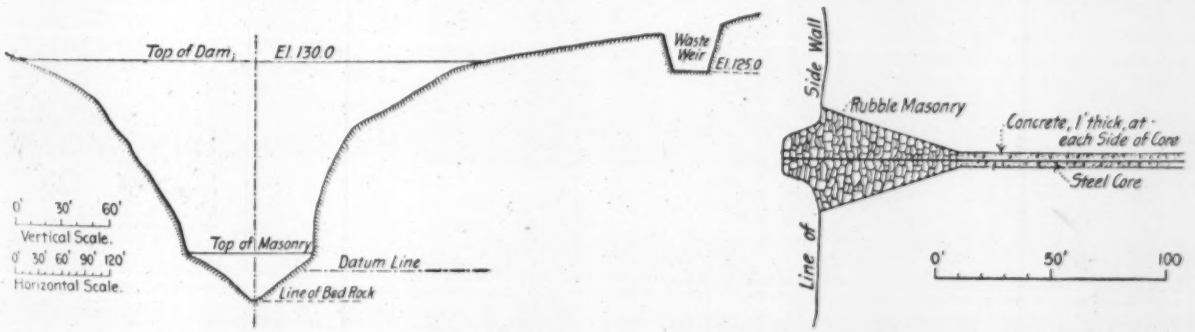


FIG. 3. PROFILE OF GORGE IN WHICH DAM IS BUILT AND SECTION SHOWING WIDENING OF MASONRY BASE ON THE LINE OF THE SIDE WALL OF THE DAM

at the time of his visit with the steepness of the downstream slope of the dam for a rock-fill structure. It appeared as if the rock had been dumped there on a slope about as steep as the material would stand. Mr. Hazen had the same impression. The rock on the downstream slope was left, apparently, just as it dropped from the cableway used in construction. Schuyler, in his "Reservoirs," states that the downstream slope is "about 1 to 1," but that the finer material on the upstream face had assumed about a 1½ to 1 slope.

The top of the dam was originally designed to be 20 ft. thick, but Schuyler states that, as first built, the thickness was about 9 to 12 ft. When the spillway was built, the material excavated was dumped on the upstream slope and the top width was somewhat increased.

Belated and Inadequate Spillway Provided

Building a dam without a spillway is a good deal like building a boiler without a safety valve, and it is a particularly noteworthy illustration of the ideas prevailing in the construction of the dam that no spillway whatever was made until the dam had been completed for two years.

The spillway is excavated through solid rock near the east end of the dam. It is 38 ft. wide and extends for 380 ft. through a rock bluff. It is lined with concrete and has a rather flat slope. The floor of the spillway is at El. 124.2 ft. above the original stream bed, and the original height of the crest of the dam was 130 ft., giving a depth of 5.8 ft. of water in the spillway before the dam would be overtopped. It is probable, however, that the crest of the dam has settled somewhat since its completion.

The sole means of drawing water from the reservoir above the dam is the outlet tunnel for the San Diego water-supply, 5 ft. in diameter, which leaves the reservoir 82 ft. below the dam crest. Nothing in the way of blowoff pipes or emergency relief outlets other than the spillway has ever been provided, and there is not even provision for drawing the city supply from different reservoir levels.

There was great delay in obtaining definite particulars as to the manner in which the dam failed, owing to the destruction of wire communications and the blocking of roads by the washing away of bridges. The

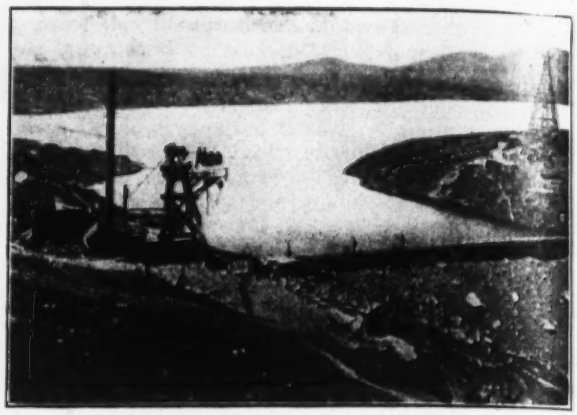


FIG. 4. VIEW OF LOWER OTAY DAM UNDER CONSTRUCTION

From "Engineering News," Mar. 10, 1898

reports of flood destruction over a widespread area of southern California, however, indicated that the rainfall must have been of an unprecedented character for that section.

The crest of the Lower Otay dam was about 480 ft. above sea level. In its flow from there to San Diego Bay, some miles distant, the flood would inevitably cut a wide channel of destruction and sweep away every dwelling in its path. The country along the lower Otay River is sparsely inhabited, or the loss of life would have been much greater.

The San Diego Water-Supply Situation

Not the least serious aspect of the disaster is the loss by the City of San Diego of its main source of water-supply, for the emptying of the Lower Otay reservoir makes it impossible for the city to draw upon the reserves of water stored in the Morena and Upper Otay reservoirs. For immediate needs, however, the city can draw about 5,000,000 gal. per day by means of its pumping plant, taking water from wells in the Mission Valley on the lower San Diego River a few miles north of the city. The remainder of the city's consumption (its total requirements are about 10,000,000 gal. per day) can probably be furnished by the Cuyamaca Water Co., which supplies water to some outlying suburbs and furnished a considerable amount of water to San Diego in 1914.

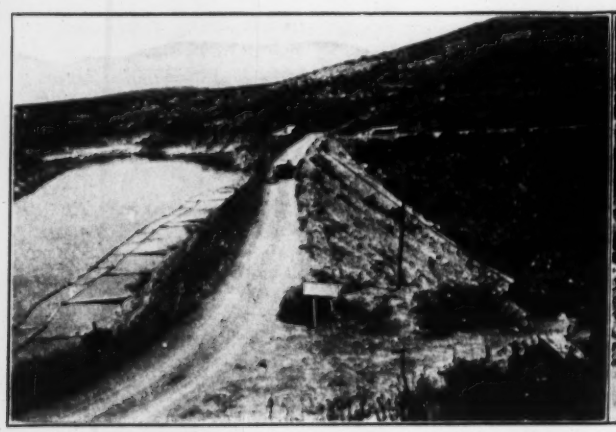


FIG. 5. LOWER OTAY DAM FOR SAN DIEGO WATER-SUPPLY ON OCT. 27, 1915

This view and Figs. 6 and 7 are reproduced from photographs taken by Allen Hazen, Oct. 27, 1915

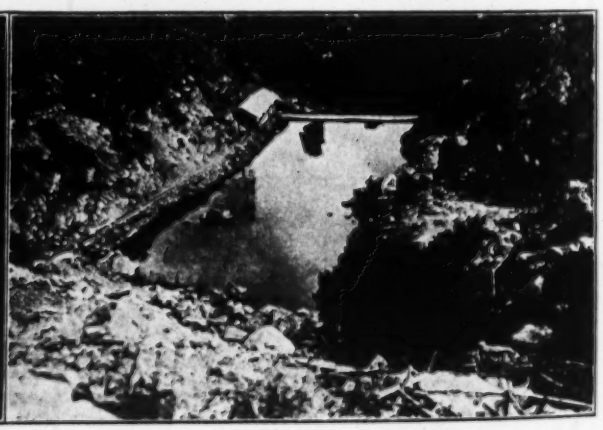


FIG. 6. VIEW OF DOWNSTREAM FACE OF LOWER OTAY DAM

Showing pool for impounding leakage and pumping house and pipe line delivering it back into reservoir

The yield of the Mission Valley wells, however, falls off as the dry season advances, and the Cuyamaca Water Co. has other imperative demands on its rather limited storage. San Diego will therefore need to proceed with the utmost dispatch to provide water from other sources. An emergency pipe line could be built from the city conduit to the Sweetwater reservoir, and probably the Otay Dam will be rebuilt at once.

Latest Account of Dam Failure

The following account of the failure of the dam was received by wire on Feb. 1 from John L. Bacon, until recently Assistant City Manager of San Diego:

The heaviest rainstorm ever recorded in this part of California caused the destruction of the Lower Otay dam at about 5 p.m. on Jan. 27. The rain began on Jan. 14 and continued for six days, with an average precipitation of 1 in. daily. Upon the ground thus thoroughly saturated a second and much heavier rainstorm fell on Jan. 25. The rain gages were inadequate to measure this storm, and the actual rainfall record at the dam is not available. The precipitation on Jan. 27, however, is estimated at about 5 in., a cloudburst occurring about 4 p.m.

The level of the water in the reservoir rose 9½ ft. from 7 a.m. to 5 p.m., at which time the dam overflowed. It began to give way about 5 min. later. The destruction was completed in 15 min., the dam being washed away to its foundations all around. The failure evidently started by the water overflowing the crest, washing out the backing of the central core. The break started near the center of the dam,

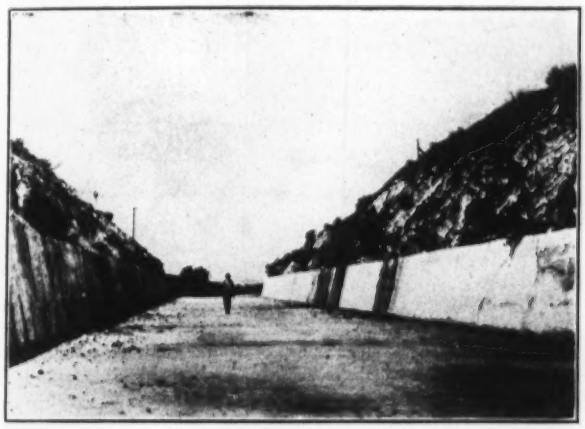


FIG. 7. SPILLWAY OF LOWER OTAY DAM FOR WATER-SUPPLY OF SAN DIEGO

the plates holding back for a time against the sides. Part of the plate was crumpled up and washed down the valley for several miles.

The floor of the spillway was at an elevation of 124 ft. above the original stream bed, and the elevation of the top of the dam was 134 ft. The spillway was 38 ft. wide at the bottom and 45 ft. wide at the elevation of the dam crest. The estimated flow through the spillway when the dam overflowed was 2,000,000,000 gal. daily. The spillway was not blocked in any way. The ¼-in. steel-plate core in the dam, with 12 in. of concrete on each side, stopped 2 ft. below the dam crest. The length of the dam crest was 560 ft. and the width of the top 16 ft. The thickness of the dam at the bottom was 400 ft. The valley below the dam was swept clear of soil to bedrock, and the banks of the valley show that a wave about 50 ft. high passed down. About 30 lives were lost in the flood.

It is estimated that to replace the dam will cost about half a million dollars. There is ample water for the city's supply stored behind several other dams. Part of the Sweetwater dam was washed out. The Upper Otay dam, an arched concrete structure of horseshoe shape, had a depth of 3 ft. of water flowing over its crest on Thursday, but is safe. The City of San Diego has been cut off from the outside world except by water. No telegraph, telephone or railway lines were in operation. All bridges were washed out, and roads were impassable.

Floods in Southern California Summarized to Jan. 25*

Beginning Sunday, Jan. 16, 1916, southern California was visited by a rainstorm the intensity of which caused high-water conditions throughout the section. The resulting floods did much damage, tying up and delaying traffic on most of the steam railroads and electric lines and rendering many families homeless. Several deaths are also attributed to it. The storm center occurred at the latitude of Los Angeles and not to the northward in the vicinity of Point Conception, as is usual.

The rainfall recorded by the United States Weather Bureau at Los Angeles, covering the four days of Jan. 16 to 19, was 6.52 in. The greatest rainfall for 24 hr. was on Monday, Jan. 17, when 4.16 in. was measured. The heaviest hour's precipitation was between 3 and 4 a.m. of that day, when 0.83 in. fell. The magnitude of the floods was largely due to the fact that during the first 15 days in January it had rained 3.29 in., which was distributed over seven rainy days, saturating the soil.

This storm is comparable with that of February, 1914, which caused severe flood conditions generally throughout southern California. The total precipitation of that storm (February, 1914) was 7.04 in., practically all of which fell in a period of three days, the greatest rainfall for 24 hr. being 4.26 in. on Feb. 18. The greatest intensity for any one hour was on Feb. 18, when 1.49 in. was recorded between 1 and 2 p.m. This 1914 storm, like the recent one, was preceded by several rainy days. The flood damage caused by the storm of 1914 was greatly in excess of that caused by the late storm.

Floods in southern California are of a "flashy" nature and emerge from the mountain cañons with great violence, carrying heavy loads of silt. Near their mountain outlets these floods erode violently. In the low lands, where the stream gradients are flat, they overflow and deposit silt on their delta cones. Large areas were flooded by the last storm to a depth of from 1 to 4 ft., and damage resulted in the vicinity of Los Angeles and Long Beach harbors from the deposit of débris.

Some of the streams were gaged during the height of the recent flood. South of Los Angeles the west flood channel of the San Gabriel River unites with the flow of the Los Angeles River and the Arroyo Seco. This combined flood flows southwesterly between the Dominguez and the Cerritos hills.

On Jan. 18, 1916, the combined flow was 31,100 sec.-ft. Of this stream approximately one-half was flowing southerly into the harbor of Long Beach. The remaining portion traveled southwesterly into the Los Angeles harbor at Wilmington, where it deposited silt to such an extent that a large ocean-going steamer in dock was grounded and the efforts of four tugs were necessary to extricate it from its position. The recent adoption of the plan for the diversion of these streams to a new outlet into the ocean, both by the army engineers and the Los Angeles County Board of Flood Control, is timely.

Measurements were made on Jan. 19, 1916, of the Los Angeles River and the Arroyo Seco above the City of Los Angeles. The maximum discharge of the Los Angeles River was roughly estimated to have been 7,200 sec.-ft., and that of the Arroyo Seco 6,200 sec.-ft. Measurement

*From the Engineering Offices of J. B. Lippincott, Central Building, Los Angeles, Calif.

was made by the United States Geological Survey of the San Gabriel River at Azusa between 11 a.m. and noon of Jan. 19, 1916, and showed a flow of 4,310 sec.-ft. The peak of the flood probably occurred on Jan. 17.

The San Luis Rey River in San Diego County discharges into the Pacific Ocean at Oceanside. This river has a drainage area of approximately 600 sq.mi. While the rainfall on the seaboard at San Diego is usually light, it is heavy in the higher mountain elevations, a rainfall of 46 in. per annum being recorded at Nellie, which is at El. 5,500 ft., approximately, on the crest of the San Luis Rey basin. At Escondido the annual precipitation is 14.89 in. Four and one-half inches of rain was recorded on Jan. 17, 1916, at the intake of the Escondido canal, at an elevation of 1,700 ft. The crest of the San Luis Rey flood of 27,000 sec.-ft. reached the mouth of the river about noon of that day. Soundings were taken from the highway bridge at Oceanside and float velocities observed. These showed during the morning of Jan. 19 a discharge of 9,500 sec.-ft., and during the afternoon 10,000 sec.-ft. The river washed out bridges at Bonsall, about 15 mi. from its mouth, and at San Luis Rey Mission, 4 mi. from its mouth. The railroad bridge north of Oceanside was washed out, and train service on the Santa Fe line between Los Angles and San Diego was suspended from Jan. 17 to 25. The Los Angeles-San Diego highway was washed out in a number of places near Los Angeles, at Cardiff and at Bernardo.

High floods occurred along the coast streams as far north as San Luis Obispo and from the high Sierras as far north as Fresno. North of this there appears to have been either less precipitation or snowfall—which does not promptly produce high water.

Drainage of Railway Roadbeds*

By M. C. Blanchard†

The importance of proper and adequate drainage for railway roadbeds has always been realized, but the rainy season of 1915, because of its length and the extraordinary precipitation, has served to emphasize that fact to those actually engaged in the maintenance of track.

Ordinarily the usual methods of drainage, such as cut and surface ditches, when properly maintained, are sufficient to carry off the surface water and provide a fairly well-drained roadbed. Frequently conditions occur, however, when these ordinary means are inadequate, and it is then necessary to provide special systems of drainage to carry away the water that sinks into the roadbed.

The most common condition requiring these special systems is the formation of a pocket or bed of porous material of considerable depth underlying the track, into which surface water sinks and is imprisoned to a greater or less degree by the adjacent earth. This condition occurs in both cuts and fills and usually is brought about in the following manner.

Formation of Ballast Pockets in Cuts

In cuts the ditches frequently become blocked by dirt that washes or slides in from the sides. The surface water is thus prevented from escaping and gradually sinks into the roadbed, making it soft and spongy. Occasionally this same condition is caused by springs or seepage occurring in deep cuts. The subgrade, thus softened, is not able to support the track and the heavy loads to which it is subjected. The result is that track and ballast are forced down, displacing the underlying earth, which heaves up in the ditches in the cut at each side. When this happens, the track is brought up to its former level on ballast, and eventually the track forces remove the displaced earth and clean out the ditches. When this is repeated year after year, the track finally becomes underlain for a depth of several feet with a material much more porous than the adjacent earth, and into this material water pours without any means of escape.

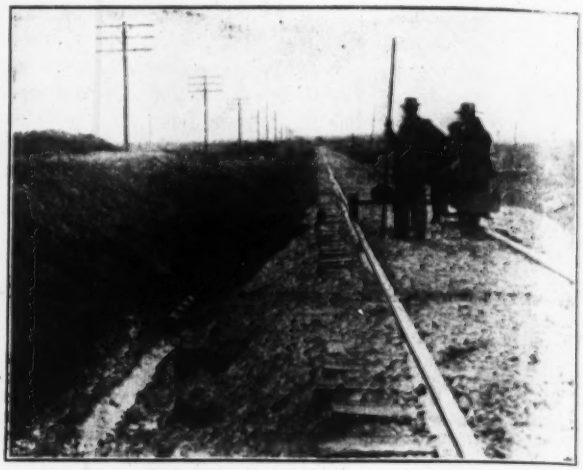


FIG. 1. TILE-DRAIN CONSTRUCTION IN CUTS

FIG. 2. BALLAST POCKET UNDER RAILWAY ROADBED. This same pocket with rock drain in place is shown in Fig. 5

*From a paper read at Kansas Engineering Society meeting, Jan. 19, 1916.

†Engineer, eastern district, Atchison, Topeka & Santa Fe Ry., Topeka, Kan.

In fills this same condition is caused in several ways. Frequently there is insufficient allowance made for the shrinkage of embankments, and the track, when it settles below the established grade, is raised on cinders or ballast. In a great many instances the original embankments are not built with slopes flat enough to prevent slides, when the filling material becomes more or less saturated under heavy rains. These slides in many cases extend back to or under the track and are temporarily repaired by dumping in cinders or ballast to the extent of making the track passable. Again this condition may be brought about by the continual raising of the track on ballast, as it is surfaced from time to time.

In all of these cases the shoulders of the embankment are usually brought up with earth secured at a time and in a manner most convenient to the track forces. The result, however, is the same as in the cut, in that the track becomes underlain with a stratum of porous material inclosed within more or less impervious earth.

The results of this condition are to make it almost impossible to maintain track in proper line and surface,

FIG. 3. TYPICAL SLIDE IN FILL, JUNE, 1915

even with a large continual expenditure of money for track labor, and in the case of embankments to cause continual slides. These slides are caused by the water collecting in the porous pockets, then seeping out through the fill, thus saturating a plane extending from the bottom of the pocket to the outer face of the embankment, upon which the superimposed material easily slips.

Remedying Pocket Conditions in Cuts

To alleviate or remedy in cuts the conditions referred to, the Atchison, Topeka & Santa Fe Ry. during the past year has been using on its eastern district a system of tile drains which may be briefly described as follows: The side of the cut is produced down to a point about 3½ ft. below subgrade to form one side of a ditch, in which is laid the main tile. The other side of the ditch is formed by a plane extending from the upper side of the end of the track tie, with an approximate slope of 1 in 1, until it intersects the plane 3½ ft. below subgrade. Using the Santa Fe standard roadbed, this results in a ditch about 3 ft. deep, 1 ft. wide at the bottom and 7 ft. wide at the top. In this ditch is placed the main drain, which usually is a 6-in. vitrified tile, with bell ends. Connected to the main by means of tees are 4-in. laterals spaced 16½ ft. apart and extending usually to a point between the rail and the center of the track. These laterals are laid so as to tap the bottom of the ballast or cinder pocket. The appearance of several of such pieces of construction may be noted in the accompanying views.

FIG. 4. ROCK DRAIN TO PREVENT REPETITION OF SLIDE SHOWN IN FIG. 3

In most instances a main drain is laid on each side of the track, and the laterals are staggered so that one lateral occurs in every 8¼ ft. of distance. The lateral trench is usually backfilled with the same material with which the track is ballasted, while the main ditch is filled with cinders. The surface is made to conform to the bottom of the original ditch. The ballast is then redressed to the standard cross-section, and the job is complete.

For draining the pockets in embankments the Santa Fe has been using trenches filled with broken stone,

FIG. 5. BALLAST POCKET SHOWN IN FIG. 2 WITH ROCK DRAIN

known in highway work as French, or rock, drains. These trenches are usually from 3 to 4 ft. wide, ordinarily at right angles to the track and extending to a depth sufficient to drain the troublesome pocket. In some cases they extend entirely through the fill, and in other cases only from about the center to one face of the embankment, depending upon the size, location and character of the pocket to be drained. The distance they are spaced apart also depends upon these same conditions and is

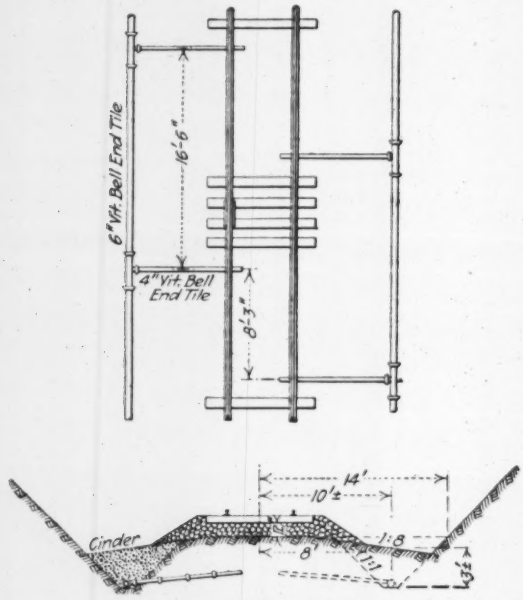


FIG. 6. DETAILS OF TILE-DRAIN CONSTRUCTION FOR DRAINING RAILWAY ROADBEDS

determined by the engineer from an examination of the ground.

The bottom of the trench is made with sufficient grade to insure the flow of water from the pocket to the face of the embankment. (This grade should be made great enough so that slight future settlement of the embankment will not destroy its effectiveness.)

The rock used for filling the trench is usually one-man-size riprap. In some of the drains that have been built, rock of this size has been carried up to within 1 ft. of the top of the trench and the remainder filled with crushed rock of the size used for ballast. This was done on the theory that the finer rock on top would prevent dirt from sifting in and gradually filling the interstices in the larger-sized rock.

There have been several places where conditions were such as to afford very poor means for carrying water away from the foot of the embankment, and in such cases a lateral drain has been constructed at the foot of the slope, into which the cross-drains connect. The lateral drain is similar in construction to the cross-drains and carries the collected drainage into the nearest ravine or depression of sufficient depth to carry the water away from the roadbed.

Problems in Draining Pockets in Fills

In places where the fill is light and the pocket or stratum to be drained is not deep—a condition that is usually caused by the repeated raising of the track in resurfacing from season to season and the consequent raising of the shoulder with earth to permit standard ballast dressing—the drains installed are ordinarily smaller than those mentioned. They are generally about 2 ft. wide and filled with spalls or rock smaller than one-man-size riprap. In other places where fills are very heavy or where sidehill construction is encountered and the drainage conditions are so bad as to make it impossible to keep the road in operation on account of slides, the drains, particularly those used as laterals, have been made much larger than any of those previously described, and they serve the purposes of a wall for bracing or retaining the roadbed, besides acting as a drain.

It has frequently been found necessary to flatten the slopes of embankments, as well as to install these drains, to prevent entirely the occurrence of slides. Very often the nature of the earth is such that it will not stand on a 1½ on 1 slope, as is usual in the construction of embankments, and in such cases the installation of drains will not wholly avert slides. On the other hand very flat slopes show a tendency to slide where porous pockets exist under the track, and therefore in such cases a combination of flat slope with rock drains should be used.

The performance of these systems of drainage has been very satisfactory. The amount of water drained from th roadbed by them for a long time after rainfall has ceased is surprising. The cost of these systems is greater than of those ordinarily used, but it is justified by the better and safer track obtained and the decreased cost of maintenance.

Why the Seattle Wood-Stave Water Pipe Failed*

By R. H. Ober†

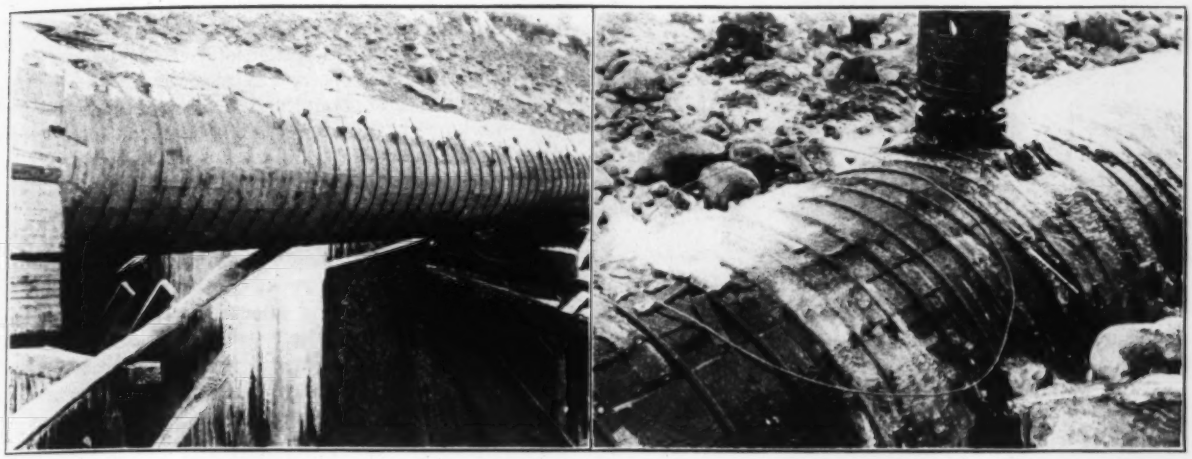
Seattle is supplied with water from Cedar River by means of two wood-stave pipe lines—Cedar River Pipe Line No. 1 and Cedar River Pipe Line No. 2. Pipe Line No. 1 was constructed in the year 1900 and put into operation on Jan. 10, 1901. Pipe Line No. 2 was constructed in the years 1908 and 1909 and was put into operation on June 21, 1909. It was in this latter pipe line that the recent breaks occurred which cut off a portion of the city's water-supply [early in January, 1916.]

The first leak appeared at Grouse Ridge, about 4 mi. from the intake at Landsburg; the second at a point near Rock Creek, about 2½ mi. below the intake; the third near the station of Noble, on the main line of the Chicago, Milwaukee & St. Paul R.R., at a point about 2 mi. below the intake; and the fourth in the vicinity of Molasses Creek, which is between Swan Lake and Renton and about 11 mi. from the intake. All of these leaks occurred in the immediate vicinity of standpipes located at summits in the pipe lines.

The first and fourth leaks were of a comparatively minor nature, involving a relatively small amount of work in the replacement of one or more staves which were not decayed, but which were defective when originally built into the pipe. The second and third leaks, however, were of a very serious nature, involving material injury to the

*A report made to the Pacific Coast Pipe Co., Seattle, Wash., which engaged the author of the report to investigate the breaks in order to learn their exact course. The author of the report was accompanied by L. M. Grant, chief engineer of the company. The company did not furnish the material for the conduit nor construct the latter.

†Consulting Engineer, Alaska Building, Seattle, Wash.; formerly City Building Inspector of Seattle.



FIGS. 1 AND 2. WOOD-STAVE PIPE LINE NO. 2, WATER-WORKS OF SEATTLE, WASH., at BREAK NO. 2, NEAR ROCK CREEK

Left—Hillside support washed from beneath pipe. Right—Portion of 150 ft. of conduit after rebuilding

pipe line and reconstruction in each case of a considerable length of pipe. The water contained in the standpipes in the vicinity of the second, third and fourth leaks had been frozen, and the standpipes had thereby been rendered incapable of performing their proper functions. The freezing of these standpipes made it impossible for air to enter the pipe lines to take the place of the water that was drawn off in cleaning the settling basin at Landsburg and in repairing the first leak at Grouse Ridge. This resulted in the formation of a vacuum in the vicinity of each of these standpipes and the consequent collapse of the pipe by reason of the unbalanced atmospheric pressure upon the outside of the pipe.

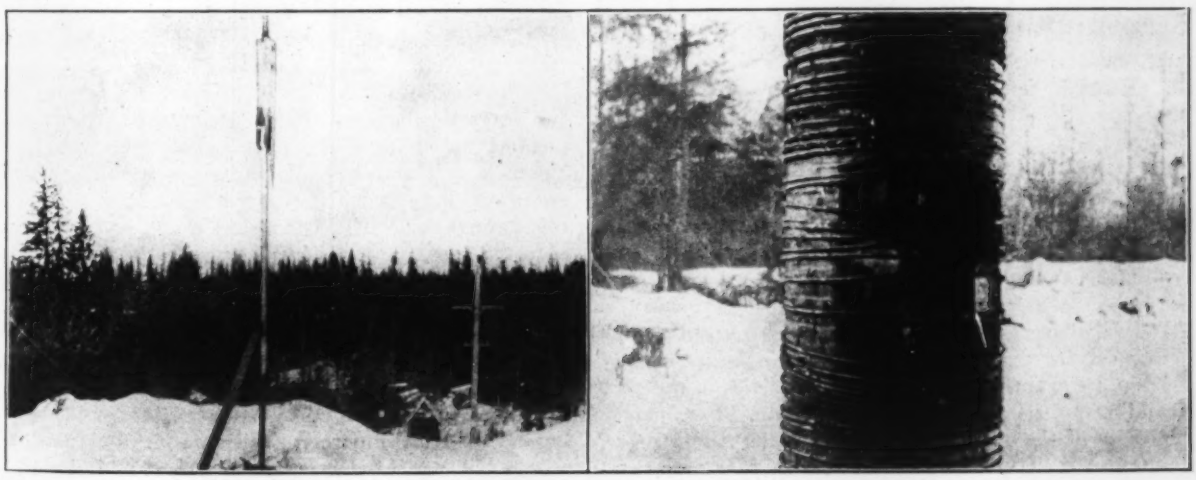
With the repair of the first leak at Grouse Ridge and the subsequent admission of water to the pipe line the escape of the water from the broken pipe in the vicinity of Rock Creek became noticeable and resulted in the washing out of the hillside at that point before the water was shut off at the intake. (See Figs. 1 and 2 for views of conduit before and after the break was repaired.)

With the repair of this section of the pipe and the subsequent admission of water into the pipe line, the damaged condition of the collapsed pipe at the third leak, in the vicinity of Noble, became noticeable through the escape of the water and the washing out at the hillside at that point. Subsequent to the reconstruction of the collapsed pipe at the third break, near Noble, the fourth break, in the vicinity of Molasses Creek, was discovered and repairs were made at this point.

It is found that the wood staves of which pipe line No. 2 is constructed, which have been in service for a period of about seven years, are in very good condition, as shown by those exposed to view and by those removed from the broken sections. The outer surface of the wood, generally speaking, shows scarcely any signs of decay; where decay is to be seen, it is relatively small in amount and extends to a depth not exceeding $\frac{3}{16}$ in. at any point observed.

It is believed that the injury to the pipe was not occasioned by the decay of the wood staves of which the pipe is constructed, nor by reason of the fact that the pipe was constructed of wood instead of some other material.

The formation of a vacuum as a result of the freezing of the standpipes would have caused the collapse of a steel pipe or of a reinforced-concrete pipe as surely as of the wood-stave pipe, and the repairs that would have



FIGS. 3 AND 4. INSULATION AGAINST FREEZING BURNED OFF STANDPIPES ON SEATTLE WATER-SUPPLY CONDUIT NO. 2

Left—Standpipe 2 mi. below intake. Right—Standpipe 11 mi. below intake

been necessary in the case of the steel or reinforced-concrete pipe would have required a much longer time and larger expense than in the case of the wood-stave pipe.

It is believed that the injury to the pipe lines and the consequent interruption of the water-supply of Seattle were due to lack of care in the construction of the standpipes and to failure to prevent the water in these standpipes from freezing.

The construction of the standpipes was defective by reason of the fact that the pipes were very much smaller than should have been installed, being, in fact, in some cases about one-seventh of the area which should be provided in order to afford reasonable protection against accidents of this kind. Lack of care was shown by the failure to protect the pipes in such a manner that there would be no possibility of the freezing of the water contained therein.

The fact that the water in those standpipes was frozen should have been discovered by the person or persons charged with the responsibility of protecting the water-supply of Seattle and should have been discovered in time to permit thawing out the standpipes before shutting off the water in the pipe lines. After the damage had occurred, it was evidently discovered that the water in these standpipes was frozen, and efforts were made to thaw out the pipes by means of building fires on the ground around them and by the use of oil applied to the standpipe within the outer casing. These fires at several standpipes destroyed what protection had been afforded by means of wooden boxes inclosing the standpipes, which boxes were filled with sawdust (see Figs. 3 and 4). The destruction of these boxes left the standpipes exposed to the weather and in a worse condition than existed before the occurrence of the breaks in the pipe lines, and they were allowed to remain in this exposed condition after the repairs to the pipe line had been completed, thus exposing the city to the danger of a further collapse of these pipes and the consequent interruption to the water-supply.

Finally, I would say that the conditions which brought about the injury to the pipe line, the consequent inconvenience to the people of Seattle and the hazard to life and property could have been avoided under a proper system of management and supervision and that without proper management and supervision similar injury would be caused, under similar conditions, to a pipe constructed of steel or of reinforced concrete.

News of the Engineering World

Civil Engineers To Vote on New Headquarters

As announced in the last issue of *Engineering News*, the Board of Direction of the American Society of Civil Engineers is sending out to the membership a circular explaining the proposal to transfer the society's headquarters in New York City to the Engineering Societies' Building on West 39th St. The board states that this question is perhaps the most important that has ever come before the society for decision.

Twelve years ago, at the time the Engineering Societies' Building was constructed, the American Society of Civil Engineers was invited to join with the three other great national societies of Mechanical, Electrical and Mining Engineers in joint ownership and occupancy of the new building. The civil engineers decided at that time not to accept the offer, but to continue to occupy their own house on 57th St., which they had built about eight years earlier.

At that time there was doubt among many members of the American Society of Civil Engineers as to the success of the scheme for accommodating the national engineering societies in a single great building. Experience has shown that most of the reasons which were urged a dozen years ago by members of the American Society of Civil Engineers against joining the other societies in the great building project are no longer controlling. The financial stability of the engineering building enterprise has been established. The 12-story building, which is now owned jointly by the three societies previously named, represents an investment of about $2,000,000. These societies own it free of all incumbrance and have in addition over $70,000 invested in a reserve fund to provide for depreciation and amortization.

The trustees who control the building for the societies have made two offers to the American Society of Civil Engineers. Under the first offer that society, by the payment of about $240,000, will become an equal partner with the others in the ownership and occupancy of the building, this sum being substantially equal to what the other societies have paid for their share of the property. Such room as the society needs will be provided on the sixth and seventh floors of the building. The sum paid by the civil engineers will be added to the present reserve fund, making a total of $330,000, in which, of course, the civil engineers would own a one-fourth interest.

The alternative offer is that an addition of 2½ stories shall be made to the top of the Engineering Societies' Building and that two of these floors shall be especially planned for the use of the American Society of Civil Engineers. The cost of this plan, it is estimated, will be no greater than that of the other plan and will not in any event exceed $250,000. The society would, as in the other plan, become an equal partner with the three other societies in the ownership and control of the building.

The libraries owned by the three other societies are already combined in one great engineering library. It is without doubt, of all libraries in the United States, the reference library most used by engineers, and through a system of correspondence it is made available for the use of engineers located anywhere in the world. It is proposed that the library of the American Society of Civil Engineers be added to this collection, which would make a total of about 150,000 engineering books. There is reason to believe that if this combination is made, a large endow-

ment will be provided to perfect further the facilities of the library for the use of engineers.

The present house of the civil engineers in West 57th St. has been appraised by experts at $325,000, of which, however, $290,000 is in the land and only $35,000 in the building, which has cost the society about $175,000. By the sale of its present property, the society could pay all the cost of its entrance to the Engineering Societies' Building, would have between $115,000 and $140,000 in its treasury and would own in addition an interest in the Engineering Societies' Building, valued at about half a million dollars.

In addition to the financial and business aspects of the proposition the Board of Direction remarks: "It is one of the purposes of this movement to strengthen the bonds of brotherhood which should exist between all engineers."

Further Notes on Kansas River Bridge Falsework Collapse

The accompanying photographs show the extent of the damage to the East Kansas Ave. bridge over the Kansas River at Kansas City, Mo. As reported in last week's issue of *Engineering News*, high water and floating ice pushed out some of the falsework bents under the suspended trusses of the bridge on Jan. 21.

The bridge was erected from the Missouri side (the far side in the views) and had been completed to the end of the cantilever arm on the Kansas side. The first three panels of the Kansas shore span had been partly erected on rather light separate pile bents under each truss when the ice flow carried away all but the two end bents, which were nearly at the shore line.

The north truss (on the left, Fig. 1) had been practically all assembled to its bent, which held sufficiently to prevent any visible damage to this truss; but the south truss had only the lower chord member resting on the falsework. This dropped 7 or 8 ft., eventually coming to rest on the two inner piles of the bent, the cap being raised from the tops of the other three piles as shown in the views. The fall of the south truss caused the loss of six stringers and a 50-ton jack, which were on the superstructure, sheared off several pin plates and badly twisted the end floor-beam and crossbracing of the two end panels. These members had not been riveted, or the damage would have been greater, as the structure dropped by shearing off the bolt heads and breaking of pin plates without distorting many of the main members.

FIG. 1. KANSAS AVE. BRIDGE AFTER WASHOUT OF FALSEWORK

Looking toward Missouri shore. North truss on left

For the week following the collapse, rain and freezing weather made it impossible to work on the structure, and to Jan. 27 nothing had been done save to assemble some groups of piles under the distorted truss. The contractor proposes to drive groups of piles under this truss, cap them and bring the truss back into line by jacks.

FIG. 2. LOOKING NORTHERLY TOWARD KANSAS AVE. BRIDGE AT KANSAS CITY

Colorado River Flood at Yuma

An unprecedented flood in the lower Colorado River on Jan. 22 did much damage in Yuma, Ariz., and vicinity; and before the levee breaks could be repaired, there were indications that a still heavier flood might come down the river.

The possibility that the Colorado might break into the Imperial Valley once more caused much anxiety, but the first of the high floods passed by without doing any such harm. A summary of flood conditions in Yuma and vicinity up to Jan. 30 has been received by wire from L. M. Lawson, project engineer United States Reclamation Service, Yuma, Ariz., as follows:

The largest recorded discharge of the lower Colorado River up to that time occurred at Yuma on Jan. 22, when the gage stood at El. 34 and the discharge was 210,000 sec.-ft., besides an additional 10,000 to 15,000 ft. passing around Indian Hill through breaks in the main canal and Southern Pacific main line. This immense river flow was due to abnormal precipitation in the drainage area of the Gila River and to slight local rains on the Colorado drainage area. The heaviest previous discharge of the Colorado River at Yuma was 150,000 sec.-ft., during the regular June rise of 1909.

The Indian Reservation levee on the California side of the river was overtopped by 12 to 18 in. of flood water, causing three openings with a combined length of 800 ft. Through this break, water was discharged by means of the main canal of the Yuma Project of the United States Reclamation Service, through the high fill of the main line of the Southern Pacific and also through the lower reservation level. The project unit on the California side of the river totals 16,000 acres, of which 10,000 was under water. Main canal banks totaling 50,000 cu.yd. and levee banks amounting to 5,000 cu.yd. were displaced.

In the Yuma Valley unit of the project only 3,000 of 50,000 acres were flooded. An unforeseen high stage of water breached the Yuma Valley levee at 4-mi. and 16-mi. points, displacing about 12,000 cu.yd. of bank.

The levee protecting the City of Yuma constructed many years ago by the War Department, but not maintained, was partly destroyed. This permitted a large discharge of water through two principal streets of Yuma, doing considerable damage. Practically all the adobe structures in the flooded area of the city were completely ruined, and some buildings were rendered unsafe.

On Jan. 28 the river had fallen to El. 18 on the gage and the discharge to 36,000 sec.-ft., but reports indicated a second rise in both the Colorado and the Gila. Tonight [Jan. 30], with the gage reading El. 30 and the river discharging approximately 146,000 sec.-ft., it is believed that an unprecedented combination of floods in both rivers is coming.

The Laguna dam across the Colorado River is uninjured, and no irrigation structures of any size have been damaged. With a few exceptions the damage to farms is slight.

In the short interval between the two floods some levee breaks were repaired. The Reclamation Service is employing a large force and extra trains. Repairs to the canal breaks will be started at once. It is expected that water service [for irrigation] will be restored in 10 to 15 days.

The Imperial Valley levee breached for about 1,000 ft. at a point 5 mi. below the Imperial Canal heading, but the Inter-California R.R. embankment has thus far prevented damage to the Imperial Valley from water passing through this break.

⌘

Decision Rendered Against Bone Cantilever Wall

A decision adverse to the Bone patent on reinforced-concrete retaining walls was rendered by Judge A. B. Anderson, of the United States District Court, in Indianapolis, on Jan. 26. The suit was brought by Frank A. Bone, the owner of United States Patent No. 705,732, against the County of Marion, Indiana, for alleged infringement of the patent in the construction of certain retaining walls by the county. The county introduced for the first time in any Bone case a number of foreign publications showing what were claimed to be anticipations of the Bone patent.

Judge Anderson gave an oral decision dismissing the complaint on the ground that the prior art disclosed walls embodying the essentials of the Bone wall. He said that the patent could obtain, if at all, only for the specific type of reinforcement shown in the application. The general design of reinforced-concrete retaining walls is a matter of longer standing than the Bone patent. The judge also gave as his opinion that the placing of reinforcing rods where tensile strain exists is not invention.

In the suit Prof. W. K. Hatt, of Purdue University, was expert for the County of Marion, and D. B. Luten, F. A. Bone, E. P. Bone and W. K. Eldridge were experts for the plaintiff.

⌘

The Final Vote on the Barge Canal referendum in New York State, excepting the vote from Oneida County, which is held up by court order, shows 615,147 votes for the $27,000,000 bond issue and 569,950 against, leaving a majority of 45,197 for the issue. The unofficial count for Oneida County showed a majority of 379 against the canal bond issue.

Maj.-Gen. Goethals Has Announced in an official statement that the Panama Canal is not ready to be reopened, nor can any prediction as to its prospective reopening be made. General Goethals says, among other things, "It is not intended to reopen the canal until a safe and practical permanent channel is reasonably assured. It is hoped that before long better information on the two main controlling features will be available. The large mass of rock at the base of Gold Hill on the east bank is being carefully observed. If the materials behind it move northerly, as is not unlikely, one danger of interruption to navigation will be removed. The other feature is the rate of motion that will result in the west slide when the dredges attack its face, as will shortly be done. As soon as this office feels that sufficient information on these two points is at hand to justify it, a prediction will be made. In the meantime it is recommended that the canal be not considered in the routing of shipping."

Heavy Rains in the Southwest on Jan. 30 and 31, particularly in Arkansas, the lower Ohio Valley and the Mississippi Valley from St. Louis to Vicksburg, have raised the rivers in the storm zone to the following critical stages, observed on Jan. 31: St. Louis, 31.2 ft., flood stage being 30 ft.; Cairo, 51.4 ft., flood stage 45 ft.; Memphis, 39.8 ft., flood stage 35 ft.; Vicksburg, 51.1 ft., flood stage 45 ft.; New Orleans, 17.8 ft., flood stage 18 ft. The Weather Bureau in commenting on this situation says: "While warnings already have been distributed to the points involved, it now seems probable that by reason of the continued rains the stages already forecasted will be exceeded slightly at various points and that high stages in the Mississippi from Cairo to the Gulf will be long continued." The West Hickman (Ky.) levee was penetrated by the Mississippi flood early on Jan. 31, resulting in the inundation of a large area in the manufacturing district. The report states that the water is backing up into the business section and that about 800 people have been made homeless. The failure of this levee is not an uncommon occurrence, owing to a sharp bend in the river at this point. Reports on Jan. 31 to the effect that the levee at Jacksonport, Ark., could not withstand the pressure of the flood waters more than a few hours more created anxiety in that locality. Trains on the Iron Mountain division of the Rock Island lines were detoured at Cypress Bayou, where the water had risen above the level of the tracks.

PERSONALS

Mr. Herschel M. Day, of Washington, Penn., has been elected Borough Engineer of Canonsburg, Penn.

Mr. John M. Egan, general manager for the receivers of the Metropolitan street Railway Co., of Kansas City, Mo., has announced his resignation, to take effect Feb. 11.

Mr. J. Lampton, recently Superintendent of the Gas Department of the Meridian (Miss.) Light and Power Co., has been appointed Superintendent of the Gas Department of the Jackson (Miss.) Light and Traction Co.

Mr. Howard P. Quick, M. Am. Soc. M. E., formerly Mechanical Engineer of the Pearson Engineering Corporation, of New York City, has opened offices as a Consulting Engineer, at 165 Broadway. He will make a specialty of public utility engineering.

Mr. William G. Muse, Civil and Mining Engineer, of West Newton, Penn., whose term of office as County Commissioner of Westmoreland County, Penn., expired on Jan. 3, has been elected Chief Road Engineer of Westmoreland County by the new Board of County Commissioners.

Mr. N. B. Rhoads, recently with the Southern Railway and Light Co., of Natchez, Miss., has been appointed Superintendent of the Jackson (Miss.) Light and Traction Co. Mr. Rhoads has had considerable experience in railway transportation in the South, and before going to Natchez he was connected with railway work in Key West, Fla., and Savannah, Ga.

Mr. Edward W. Cunningham, M. Am. Soc. C. E., for the past four years Assistant Building Inspector, Department of Public Safety of the City of Cleveland, Ohio, has been appointed Commissioner of Buildings in that city. Prior to his employment with the City of Cleveland, Mr. Cunningham was First Assistant Engineer of the Osborn Engineering Co.

Mr. Frank E. Weymouth, M. Am. Soc. C. E., recently Supervising Engineer of the United States Reclamation Service on the construction of the Arrowrock dam at Boise, Idaho, has been appointed Chief of Construction of the United States Reclamation Service, with headquarters at Denver, Colo. He succeeds Mr. Sydney B. Williamson, who has resigned to accept private employment. Mr. Weymouth has been in the service for about 13 years in construction an doperation fields.

Mr. Ray S. Blinn, for the past five years City Engineer of Mt. Vernon, Ohio, has been elected Manager of the Town of Westerville, Ohio, with a salary of $1,500 a year. Mr. Blinn graduated from Ohio State University in 1893 and for some years after graduation was in the service of the Illinois Central R.R. For three years he was Division Engineer of the company at Chicago. Later he was connected with the city engineering department of Columbus, Ohio, in charge of paving, but on account of ill health was obliged to give up his work there.

Mr. John E. Dixon, Assistant Manager of Sales of the American Locomotive Co., has been appointed Vice-President in Charge of Sales of the Lima Locomotive Corporation, with headquarters at 50 Church St., New York City. Mr. Dixon is a graduate of the University of Wisconsin in mechanical engineering. In 1900 he entered the shops of the Brooks Works, at Dunkirk, N. Y., and in 1905 was made a salesman, and later manager, of the Atlantic Equipment Co., a subsidiary of the American Locomotive Co. In 1907 he was appointed Assistant Manager of Sales of that company.

Mr. Frank S. Krug, City Engineer of Cincinnati, Ohio, has been appointed chief engineer of the proposed Cincinnati Rapid Transit and Interurban Ry. He entered the office of the county engineer of Hamilton County, Ohio, upon graduation in civil engineering from the Pennsylvania Military College in 1884, and in 1891 was appointed County Engineer, in which office he remained for 14 yr. For some years afterward Mr. Krug was Secretary and Chief Engineer of the Kirchner Construction Co., of Cincinnati, and in 1914 accepted the appointment of City Engineer of Cincinnati, succeeding Henry M. Waite.

Mr. Samuel W. Fleming, Jr., of Harrisburg, Penn., has become a member of the firm of Farley Gannett, Consulting Engineer, Telegraph Building, in that city. Mr. Fleming for the last few years has been engaged in making reports on, and investigations of, electrical properties in the East and the Middle West and on electric railways in New York and Maryland. He will devote himself to the electrical work of the firm, giving especial attention to the design and construction of electric properties for corporations and municipalities and to valuations for rate adjustments; while the operation of properties will also be undertaken for corporations and municipalities. Mr. Fleming graduated from Princeton University in 1906 and then studied engineering at Lehigh University, graduating from that institution in 1909 with the degree of mechanical engineer. For several years afterward he was identified with the Central Hudson Gas and Electric Co. as Power Engineer.

OBITUARY

E. D. Barrett, a civil engineer employed on the new Peekskill (N. Y.) aqueduct of the Catskill Water-Supply, was shot and killed a few days ago by an Italian gardener who lived in the vicinity. Mr. Barrett was 61 years of age.

William Guiler, President of the William Guiler Co., of New York City, died at his home there on Jan. 18, at 45 years of age. Mr. Guiler built the first railroad in Porto Rico and erected the two wireless towers at Santo Domingo for the Government. He was well known in South America, where for a number of years he executed large contracts for coffee and sugar machinery, railway supplies and bridge materials. He was born in Scotland, the son of Hugh Guiler, a prominent naval architect.

Albert H. Scherzer, President of the Scherzer Rolling-Lift Bridge Co., of Chicago, was killed, on Jan. 28, by falling down an elevator shaft in the Monadnock Block. He was born in Peru, Ill., in 1865, and graduated from the Technical School of Zurich, Switzerland, and the Union College of Law, in Chicago. He practiced law for some years, but on the death of his brother, William Scherzer, the inventor of the rolling-lift type of bascule bridge, he became President and active head of the bridge company.

ENGINEERING SOCIETIES

COMING MEETINGS

INDIANA SANITARY AND WATER-SUPPLY ASSOCIATION.
Feb. 2-5. Annual meeting in Indianapolis. Asst. Secy., W. F. King, Indianapolis, Ind.

INDIANA ENGINEERING SOCIETY.
Feb. 3-5. Annual meeting in Indianapolis. Secy., Charles Brossmann, 1616 Merchants Bank Bldg., Indianapolis, Ind.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Feb. 4. Mid-winter meeting in Chicago. Secy., E. B. Burritt, 8 W. 40th St., New York City.

OREGON SOCIETY OF ENGINEERS.
Feb. 7. Annual meeting. Secy., H. M. Morse, Portland, Ore.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
Feb. 8-9. Mid-winter meeting in New York City. Secy., F. L. Hutchinson, 29 W. 39th St., New York City.

OHIO ENGINEERING SOCIETY.
Feb. 9-11. Annual meeting in Columbus. Secy., John Laylin, Norwalk, Ohio.

MINNESOTA SURVEYORS AND ENGINEERS SOCIETY.
Feb. 10-12. Annual meeting at St. Paul. Secy., Geo. H. Herrold, St. Paul, Minn.

ILLUMINATING ENGINEERING SOCIETY.
Feb. 10-11. Mid-winter convention in New York City. Secy., A. S. Miller, 29 W. 39th St., New York City.

NINTH CHICAGO CEMENT SHOW.
Feb. 12-19. First Infantry Armory and Coliseum. Secy., R. F. Hall, 208 South La Salle St., Chicago, Ill.

CONNECTICUT SOCIETY OF CIVIL ENGINEERS.
Feb. 14-15. Annual meeting at New Haven. Secy., J. Frederick Jackson, Chamber of Commerce Bldg., New Haven, Conn.

BUILDING INSPECTORS' CONFERENCE.
Feb. 14-15. In Chicago. Secy., S. J. Williams, Madison, Wis.

AMERICAN CONCRETE INSTITUTE.
Feb. 14-17. Annual convention in Chicago. Secy., John M. Goodell, 1418 Walnut St., Philadelphia, Penn.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Feb. 14-17. Annual meeting in New York City. Secy., Bradley Stoughton, 29 W. 39th St., New York City.

ASSOCIATION OF ONTARIO LAND SURVEYORS.
Feb. 15. Annual meeting in Toronto, Can. Secy., L. V. Rorke, Parliament Building, Toronto, Ont.

SECOND NATIONAL CONFERENCE ON CONCRETE ROAD BUILDING.
Feb. 15-18. At Chicago. Secy., J. P. Beck, 208 South La Salle St., Chicago, Ill.

IOWA ENGINEERING SOCIETY.
Feb. 16-18. Annual meeting at Des Moines. Secy., J. H. Dunlap, Iowa City, Iowa.

NEW ENGLAND ASSOCIATION OF GAS ENGINEERS.
Feb. 16. Annual meeting in Boston. Secy., N. W. Gifford, 38 Central Sq., East Boston, Mass.

AMERICAN CONCRETE PIPE ASSOCIATION.
Feb. 17-18. Annual convention in Chicago. Secy., E. S. Hanson, 538 South Clark St., Chicago, Ill.

IDAHO SOCIETY OF ENGINEERS.
Feb. 21-23. Annual meeting in Burley. Secy., I. F. Shaffner, Boise, Idaho.

ARKANSAS ENGINEERING SOCIETY.
Feb. 22-23. Annual meeting at Helena. Secy., W. J. Parkes, Citizens Bank Building, Pine Bluff, Ark.

AMERICAN ROAD BUILDERS' ASSOCIATION.
Feb. 22-25. Annual convention in Pittsburgh. Secy., E. L. Powers, 150 Nassau St., New York City.

The Utah Society of Engineers will hold its annual meeting in Salt Lake City on Apr. 21. The secretary is C. J. Ullrich, State Engineer's Office, Capitol Building, Salt Lake City.

The Association of Iron and Steel Electrical Engineers announces that the 1916 convention will be held in Chicago, Sept. 18 to 22. The secretary is W. O. Oschmann, Oliver Iron and Steel Co., Pittsburgh, Penn.

The Memphis Engineers' Club at the recent annual meeting elected the following officers: President, C. K. Chapin; vice-president, A. L. Dabney; secretary, C. C. Pashby; directors, J. H. Haylor and H. N. Howe. The club membership is now 40.

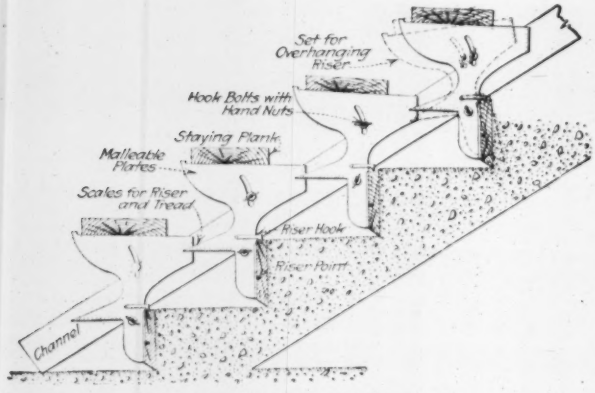
Engineers' Club of Atlantic City—Civil, electrical, mechanical, sanitary and hydraulic engineers and architects and structural contractors of Atlantic City, N. J., and vicinity have formed an engineers' club. This is said to be the first attempt to form such an association in this locality. Mr. J. Willis Gale, the secretary, states that the present outlook is for a large and energetic organization.

The Montana Institute of Municipal Engineers held the fourth annual meeting at Billings, Mont., Jan. 17 to 19. On the opening day two papers were presented: "Street Paving," by W. B. Vestal, Jr., and "Improvement District Laws." These were discussed by J. N. Edy, H. M. Sprague and E. S. Severance. On Tuesday morning Prof. Leon D. Conkling, of the Montana State College, delivered an illustrated address on "State Highways." Then there were moving-picture talks on "The Manufacture of Pipe," by a representative of the National Tube Co., and "The Concrete Farm," by a representative of the Red Devil Cement Co. In the afternoon Doctor Koyl, water expert for the Great Northern Ry., spoke on "Water Softening and Purification," while Mr. Davidson, of the Montana State College, delivered an address on "Standard Methods for Testing Sewage and Sewage Effluents." Papers were read by M. L. Morris, on "Parks and Playgrounds"; by State Horticulturist Dean, on "Preservation of Trees"; by G. C. Bruett, on "The Various Methods of Sewage Disposal, with Approximate Cost." These addresses were all well received and provoked much discussion, and this session was the strongest and most technical of the entire meeting. On Wednesday morning, F. C. Snow spoke on "Activated-Sludge Process of Sewage Treatment," while Chief Engineer Smith, of Burns & McDonnel, of Kansas City, gave an illustrated talk on "Water-Works Installations." In the afternoon the new filter plant of the Billings water-works was inspected. The officers elected for 1916 are as follows: President, Carl C. Widener, Bozeman, Mont.; vice-president, G. W. Helmick; secretary, John N. Edy; directors, M. L. Morris and F. C. Snow. The membership list now numbers 30 active, two honorary and two fellow members.

Appliances and Materials

Adjustable Step and Stair Form

A simple and novel concrete step and stair form has been devised by H. L. Park, of Reading, Mass., and is being sold by the H. L. Park Co., of that city. As shown by the accompanying sketch, the form consists of two steel channels supporting a suitable number of malleable-iron side plates which



PARK STEP AND STAIR FORM

carry the riser boards. The plates are held against the channel by hook bolts and hand nuts and are readily adjustable for height of riser and width of tread. Scales marked on the plates make these adjustments easy. A shoulder on the back side of the malleable plates keeps them in line on the channel. The riser boards are fastened to the plates by driving them on two steel points and clamping with top hooks. The plates can be adjusted for an overhanging riser. The upper part of the plates carries staging boards for workmen to go up and down. It has been found very easy with these forms to make steps that line up perfectly.

This form was developed first for long reinforced-concrete stairs, but it has been found equally satisfactory for short steps. For small contractors, short 2x4 joists are used in place of the steel channels. In service the channel or joist is first fastened at the top or bottom, the plates set and the riser boards put on.

* * *

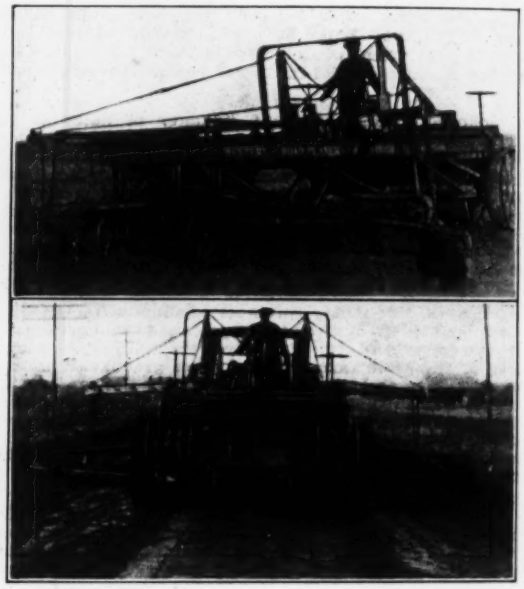
Trailer for Touring Cars

Under the name of "Troy Junior" a trailer wagon, designed particularly for use behind touring cars, has been developed by the Troy Wagon Works Co., Troy, Ohio. The track is 56 in. and the wheelbase 70 in. Wheels are of 36-in. diameter and have solid rubber tires, roller bearings and 1¼-in. square solid steel axles. The body is 35 in. wide, 96 in. long and 10 in. deep, with 6-in. side flare boards and drop gates at each end. The front springs are transverse semielliptic; the rear are full elliptic. A cushioned spring drawbar has its draft off the front axle and guides the front wheels in the track of the car. This trailer weighs 500 lb. and can carry 1,250 lb. The price is $125.

* * *

Road Planer and Finisher

A new road planer and finisher, designed to resurface, roll and finish the roadbed all in one operation, is shown in the accompanying views. The main frame is built of 10-in. I-beams. The wheels are 24-in. front and 48-in. rear, all with 8-in. tires. The tracking width of the front wheels is 5 ft. 2 in., and that of the rear wheels 8 ft. 9 in. The machine trims a 30-ft. width. Each blade, or scraper, is made in three 8-ft. sections, the rear sections on each side being fastened together. Each section of the blades is fitted with



WESTERN ROAD PLANER AND FINISHER

a detachable share like that of an ordinary road machine. Mold boards and shares are all interchangeable. There is a scarifier attachment between the blades, which is used as a supplementary leveling device. This is made of structural steel with tool-steel teeth and is drawn independently of the scraper blades and roller by direct connection to the king-bolt. It can be automatically set to cut to any desired depth. Between the rear wheels are six 42-in. rollers with a 12-in. face.

The control of the machine is as follows: The forward end of each blade is carried on a chain, going to a swinging drum attached to a crane arm. The chains are wound up by a worm gear and handwheel. The middle and rear blade sections are controlled by levers. These blades are for leveling and smoothing only, as the scarifier preceding them does the cutting. A supplementary leveling blade is hinged to the rear of the side blades to prevent leaving a sharp ridge at the center of the road. This device is controlled by a separate lever. The roller attachment is thrown in or out by self-locking worm gears operated by two handwheels, one on each side of the main frame.

It is claimed that this outfit can be used effectively for resurfacing hard roads as well as dirt and gravel. The machine is made by the Western Wheeled Scraper Co., Aurora, Ill.

Encyclopedia em Portuguez de Machinas para Construcções, Pagina 118

Engineering News

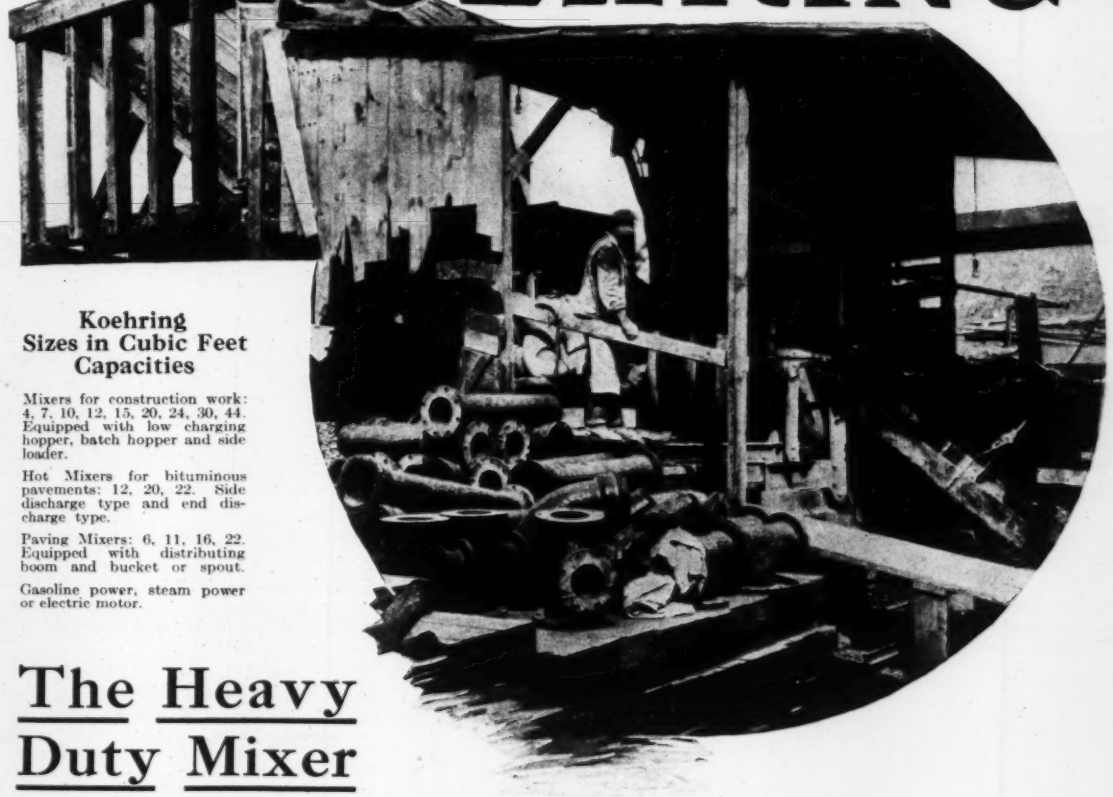
Volume 75, No. 6
Issued Every Thursday
Hill Publishing Company

NEW YORK, FEBRUARY 10, 1916

Price, 15 Cents
Contents, First Page
Advertising Index, Last Page

PERIODICAL ROOM RECEIVED FEB 1916 UNIV. OF MICH. LIBRARY

KOEHRING



Koehring Sizes in Cubic Feet Capacities

Mixers for construction work: 4, 7, 10, 12, 15, 20, 24, 30, 44. Equipped with low charging hopper, batch hopper and side loader.

Hot Mixers for bituminous pavements: 12, 20, 22. Side discharge type and end discharge type.

Paving Mixers: 6, 11, 16, 22. Equipped with distributing boom and bucket or spout.

Gasoline power, steam power or electric motor.

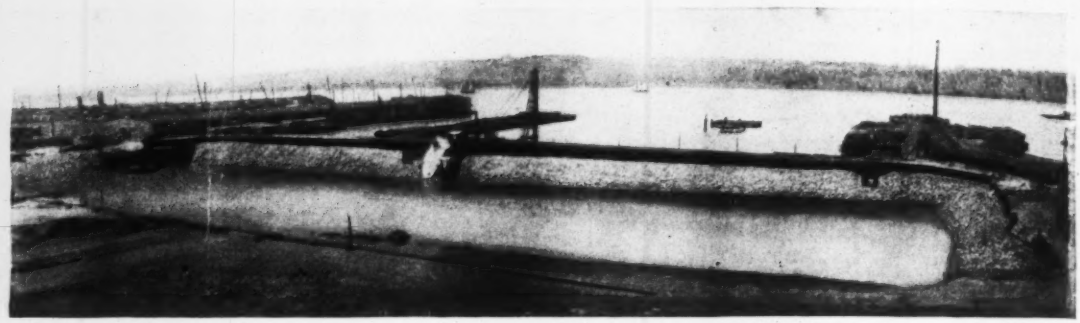
The Heavy Duty Mixer

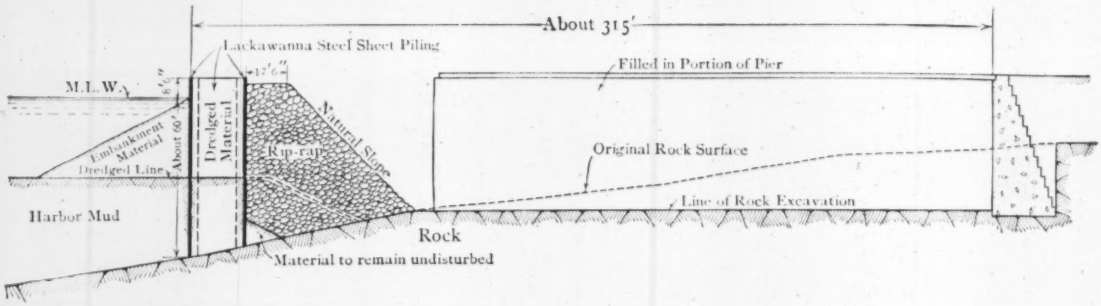
Built with a generousity of material and liberality of design, the Koehring is, beyond question **the heavy duty mixer** from which you can demand the utmost capacity, in continuous, season-after-season operation—the mixer which is a **safety-factor** against delays and breakdowns—**insurance** that the job will go through on schedule. Every Koehring, from the four-foot capacity to the forty-four-foot capacity has heavy-duty construction.

Investigate the trucks of the Koehring, the main frame, the drum suspension, the superframe, the drive, the bearings, and you can see what Koehring heavy-duty construction means to you in reliability and durability.

Write for five booklets giving details of Koehring heavy-duty construction.

KOEHRING MACHINE CO., **Milwaukee, Wis.**





Cofferdam for 46th St. Piers, North River, New York City

The deepest and one of the largest steel sheet piling cofferdams on record. Lackawanna Steel Sheet Piling, as usual, was used exclusively.

Built Under Direction of Dept. of Docks and Ferries, New York City

R.A.C. Smith, Commissioner of Docks, R.C. Harrison, First Deputy Commissioner, C. W. Staniford, Chief Engineer, R.T. Betts and J.J. Pernoff, Ass't Engineers. Contractors; Holbrook, Cabot & Rollins Corporation, New York. Thos D. Bryson, Engineer in charge, Daniel Hughes, Superintendent.

The design and execution of this dam were under the direction of C. W. Staniford, Chief Engineer, and his assistants.

Most prominent among the proposed new docking facilities on the Manhattan side of the North River are a 1000-foot pier 150 feet wide, at the foot of 46th Street and having wide slips of equal length and excavated to a depth of 44 feet on each side: also a half pier 1000 feet long extending along the northerly side of 44th Street and having a similar 1000-foot slip on the north side.

At this location in the North River, the pier-head line is 700 feet riverward from the old bulkhead line, so extension of the old bulkhead line inshore 300 feet was necessary to provide for 1000-foot docks. The slips had to be excavated at the shore end through a ledge of rock running from 20 feet in depth near the bulkhead line to zero at about 250 feet from that line.

Nearness of rock to the surface at the shore end suggested the construction there of a filled-in quay-wall for a distance of 219 feet and from there on for a 781-foot river end of the ordinary timber pile construction

The importance of this work to engineers interested in pier foundation and other subaqueous work of unusual size cannot be overestimated. We shall therefore devote our advertising space here for some weeks to the more important features. If for no other reason than keeping yourself well posted, you should watch for *and read* this series. All managing and contracting engineers should also have a copy of our book "Lackawanna Steel Sheet Piling."

LACKAWANNA STEEL COMPANY

General Sales Office and Works: Lackawanna, N. Y.

NEW YORK
BOSTON
BUFFALO
PHILADELPHIA
CLEVELAND
CINCINNATI
CHICAGO
DETROIT
ST. LOUIS
ATLANTA
SAN FRANCISCO

Licensees for the manufacture of Lackawanna Steel Sheet Piling:

For Great Britain and British Colonies in the Eastern Hemisphere, Cargo-Fleet-Iron Co., Ltd., Middlesborough, England. For France, Italy, Spain, French Colonies and Protectorates, Italian Colonies and Spanish Colonies in the Eastern Hemisphere: Cie Des Forges & Aciéries de la Marine et d'Homécourt, Paris, France.

225

Engineering News

VOLUME 75 FEBRUARY 10, 1916 NUMBER 6

Concreting Plant for Halifax Ocean-Terminal Work

[With Inset Sheet]

The new Ocean Terminals now under construction at Halifax, N. S., were described in *Engineering News*, Dec. 9, 1915, p. 1118. The plan contemplates an entirely new railway entrance to the city and the construction of a 2,000-ft. landing quay and six 1,200-ft. piers providing water at depths from 30 to 45 ft., with a complete freight terminal and a passenger railway station in immediate conjunction with the docks. At present the railway cut, a contract of about $1,750,000 is near completion, and work is well under way on the first stage of the dock work. This comprises the landing quay and one of the piers and the breakwater, which protects the whole pier section. This dock work is held by one contract, which will total about $5,500,000. The contractor, Foley Bros., Welch, Stewart & Fauquier, has been engaged about a year and a half. The work to be done comprises the manufacture of a great number of large reinforced-concrete hollow blocks that go to make up the quay wall, the placing of these blocks and the dredging of the harbor to the required depth. For this work a remarkably large and well-equipped plant has been installed. Fig. 3 shows a plan layout of the terminal site and the contractor's plant. The large photograph in the inset sheet that accompanies this issue of *Engineering News* is a panorama view taken just inshore of the contractor's plant, overlooking that plant and Halifax harbor. The various parts of the plant are shown both in the view and in the plan.

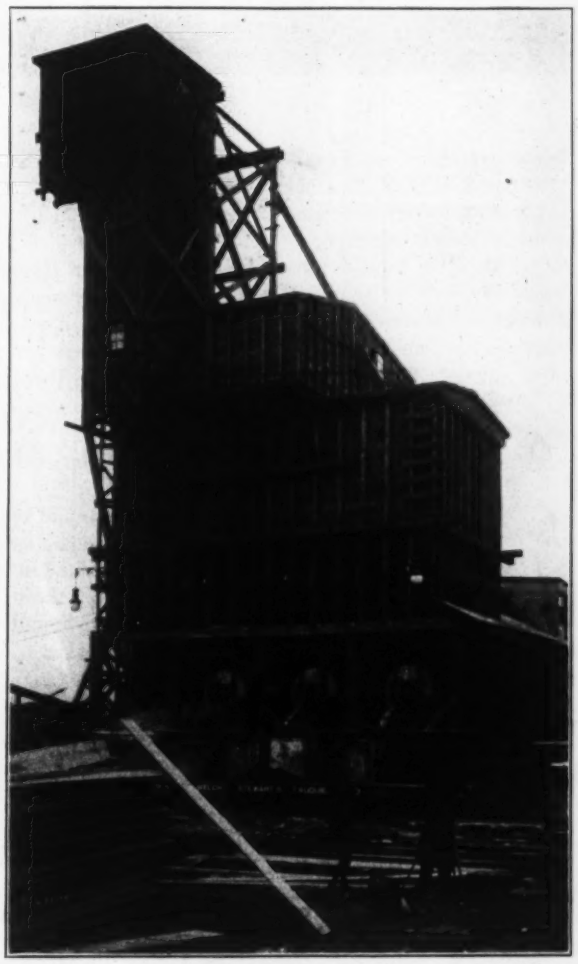


FIG. 1. CONCRETE-MIXING PLANT AT NEW HALIFAX OCEAN TERMINALS

The details of the railway contract work were described in the issue of Jan. 27, 1916, p. 188, and the subaqueous excavation, Feb. 3, 1916, p. 228. The present article will be devoted to the manufacture and handling of the large concrete quay-wall blocks. The very novel method of laying up these blocks in the wall was described in the issue of Dec. 9, 1915. In brief, these walls are made up of rows of huge concrete blocks set up in tiers to a height of 50 to 60 ft. and footing on concrete pedestals set inside a diving bell on the dredged rock bottom. Each block has cells which are filled, after placing, with concrete or sand and stone, the concrete filling being along the front and in the center transverse row of cells in each block. The blocks vary in dimension with their location in the wall, but for the most part are of one standard size, the variation from that size being relatively slight. The standard block is 31 ft. long, 21 ft. 10 in. wide and 4 ft. deep. It has 8-in. walls reinforced for both permanent and handling stresses and is provided with 15 open cells. Each block contains about 30 cu.yd. of concrete and 5,000 lb. of reinforcement and weighs 60 tons. Of the 3,634 blocks to be used, over two-thirds are of this type. The concreting plant is arranged so that the material enters at the south end of the site and moves during the manufacturing process toward the north, where the blocks are laid. The sand and gravel, which are excavated from a combined pit about 14 mi. away, are brought by train and by barge across Halifax harbor to the wharves immediately alongside of the concrete plant. They are dumped in piles over a conveyor tunnel that runs from the crusher to the mixer house, using both floating and railway cranes.

FIG. 2. LOOKING WEST OVER BLOCK-CASTING YARD AT THE

The details of the mixer house are shown in Fig. 4. The general route followed by the material from the stock pile to the mixer is as follows: The combined sand and gravel, containing considerable large material, is dumped over the conveyor tunnel in high storage piles and is let through hoppers onto the belt as required. The belt carries it to the elevator at the house, which passes it onto screens that separate the fine and coarse aggregate into the bins over the mixers. The oversizes pass through an open chute to a 12x24-in. Reliance jaw crusher, which in turn passes the reduced sizes back to the elevator and thence to the bins.

At the north end of the conveyor tunnel there is another jaw crusher, which was installed to crush the larger stones from the storage pile. As at present operated, these stones are picked from the conveyor belt and wheeled to the north end of the tunnel, where they are thrown into the crushers. From there the reduced sizes are elevated and thrown onto the storage pile again. This has not proved a satisfactory method of caring for the large stone, and next season an effort will be made to get rid of the large stones before they reach the storage pile.

The mixer plant is equipped with four 30-cu.ft. (loose measure) Ransome mixers that are motor driven from electric power furnished by the near-by power house. Originaly there were three standard yard mixers in the mixer house, but it was found that the batch so delivered was

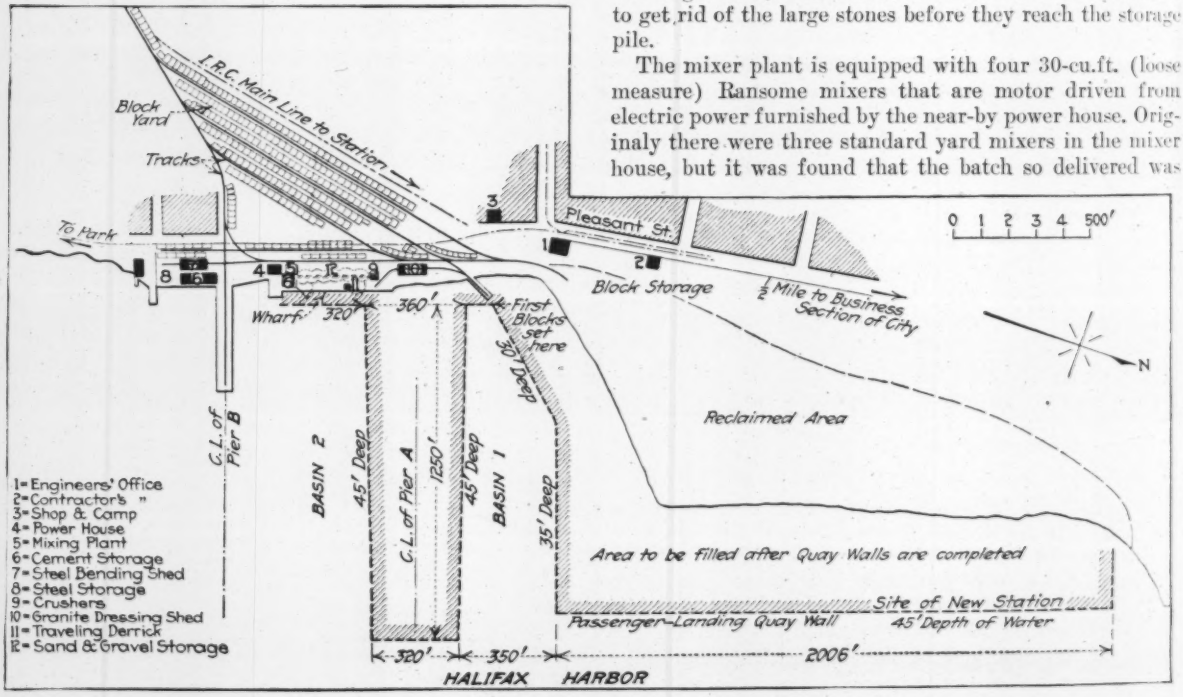


FIG. 3. LAYOUT OF CONTRACTOR'S PLANT AT HALIFAX OCEAN TERMINALS

HALIFAX OCEAN TERMINALS; RAILWAY APPROACH AT EXTREME LEFT

too great to be conveniently distributed in the block forms, so the drums were changed to a special 30-cu.ft. size and another mixer installed. This plant has a capacity of about 1,000 cu.yd. per day. The concreting plant was installed by the Ransome Concrete Machinery Co.

Layout of Concrete Block Yard

In front of the mixer house a block yard had been laid out with standard-gage railway track on the ordinary ladder-track design. It is so arranged that the platforms for the concrete-block forms can be placed with two lines of forms to each intervening space between tracks. A general view of the yard, looking west from the concrete mixer house, is shown in Fig. 2.

A number of methods of conveying the concrete from the mixer to the form were considered, including spouting from a centralized plant and spouting from localized plants to which concrete would be brought in by train. After some experimentation and study, it was decided most economical to convey the concrete from the mixer to the form in buckets resting on cars in small trains. Such a train is shown in Fig. 6.

The blocks are formed on wooden platforms with forms furnished by the Blaw Steel Construction Co., Pittsburgh. The details of one of the standard blocks as it

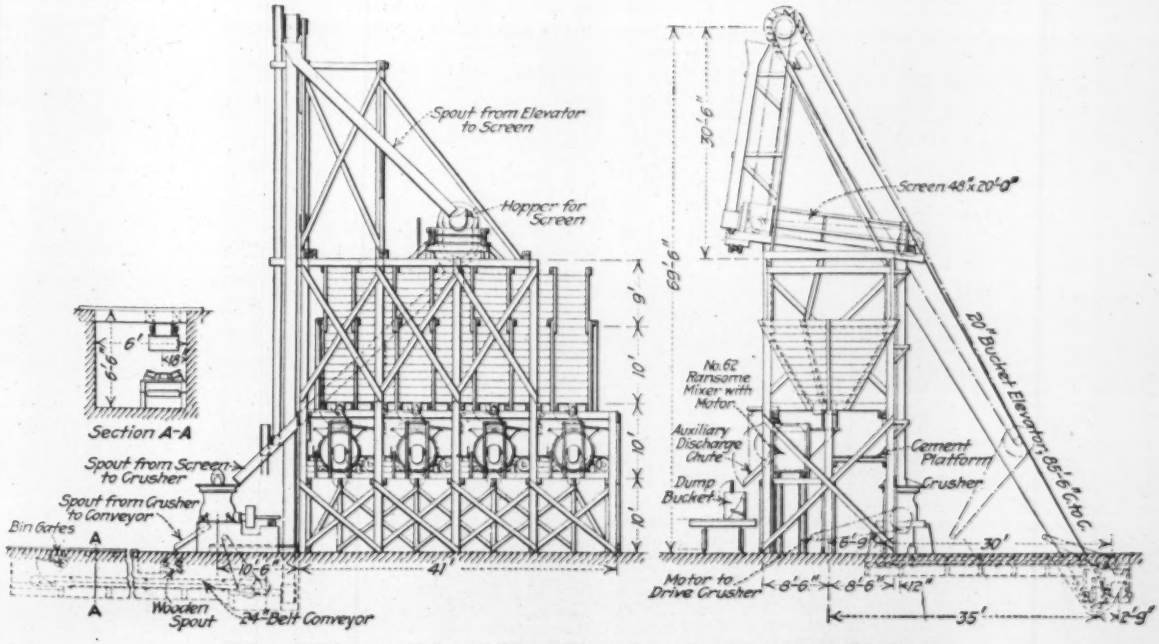


FIG. 4. DETAILS OF CONCRETE-MIXING PLANT AT HALIFAX HARBOR

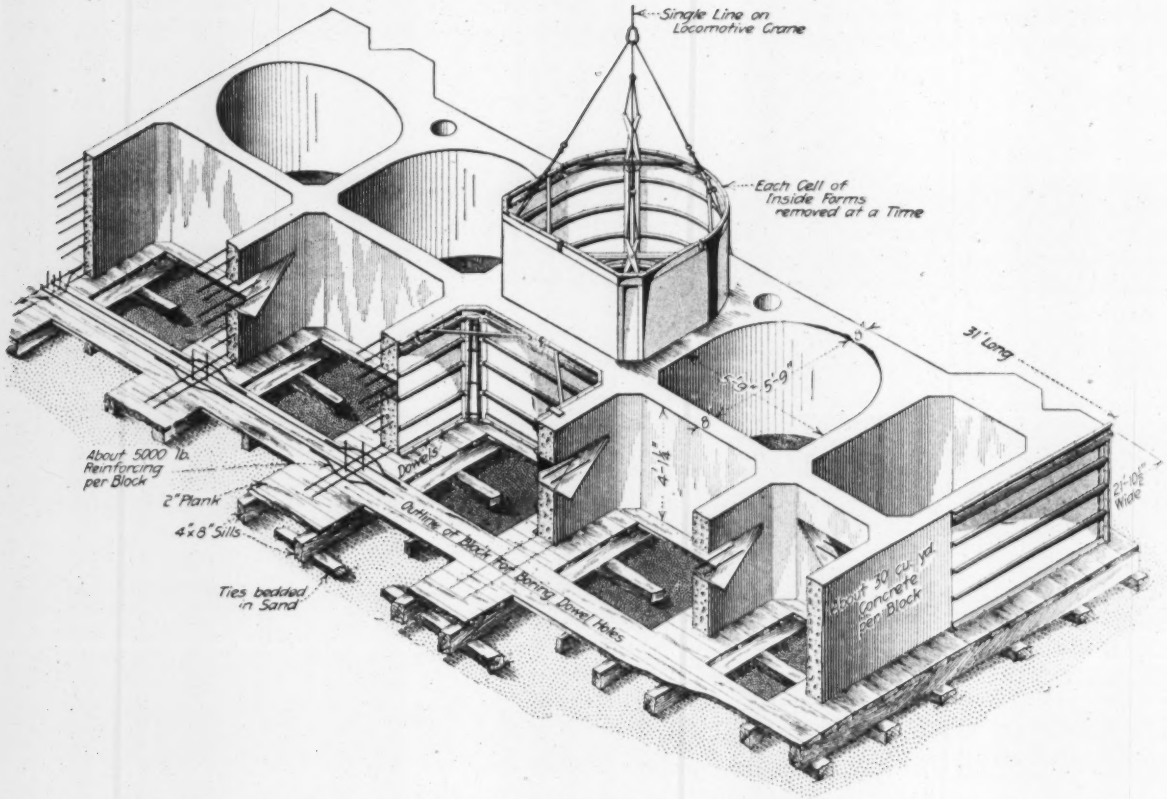


FIG. 5. PERSPECTIVE VIEW OF CONCRETE BLOCK IN PROCESS OF MANUFACTURE, USING BLAW STEEL FORMS

is formed in the yard are quite clearly shown in the perspective in Fig. 5.

The casting platforms are made up by bedding 8x10-ft. ties in the sand and placing thereon 6x8-in. sills, to which is spiked a 1¾-in. floor. On this floor the outside steel forms, of outside-flanged steel plates, are held in place by dowels inserted through the bottom flange of the form and into the floor, with the inner face of the form fitted along lines marked on the floor. The tops of the forms are spaced and held in position

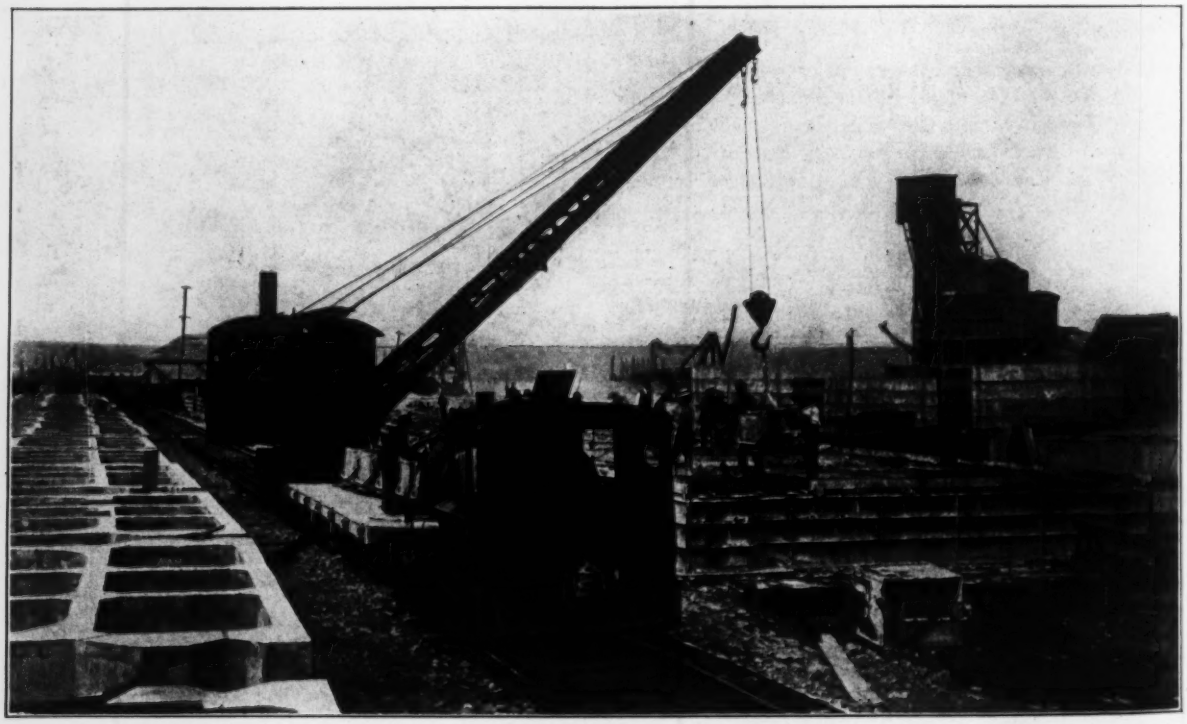


FIG. 6. CONCRETING THE BLOCKS FOR THE QUAY WALL AT HALIFAX

by flat spacers set above the top channels of the forms, thus making it unnecessary to put any tie-rods through the concrete.

On account of the amount of steel in each block, it is necessary to set the forms and the steel together. The practice is therefore to set the forms for the three interior pockets first, and the bars for the inner walls are then placed and wired in position around these forms. After this the forms for the outside pockets with their reinforcement are set, and then follows the placing of steel for the outside walls, after which the outside forms are set up.

The steel forms are made up of light plates reinforced with angles. In Fig. 5 is shown the form for one of

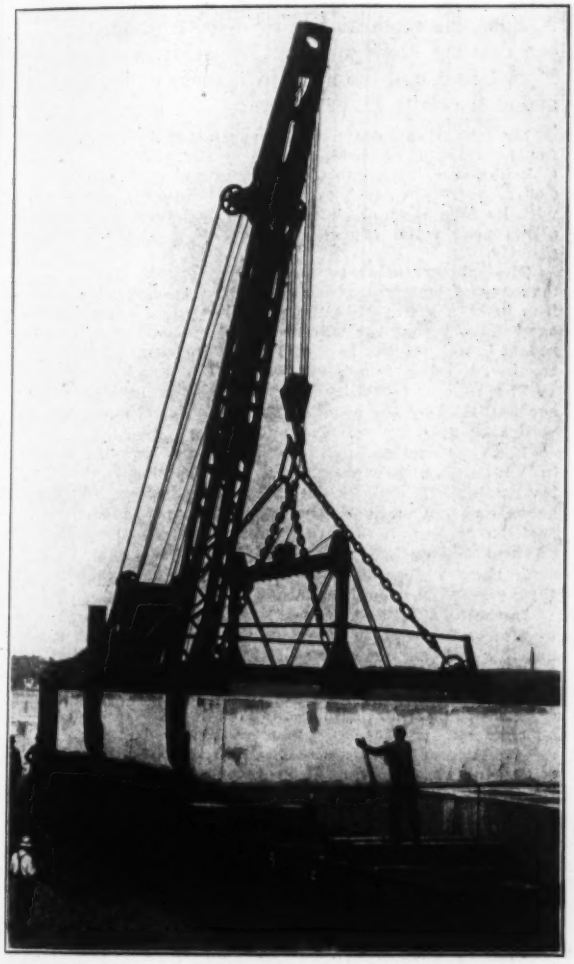


FIG. 7. 160-TON CRANE HANDLING 60-TON BLOCK

the pockets. It consists of four side plates and four corner channels bolted together, so that by the removal of slotted bolts which lock the plates and channels together the entire form can be collapsed and withdrawn in one piece. A lifting gear with four tongs engages the plates at the corners.

These forms are set up on the platforms, which are fixed in groups of eight to ten, and each group is concreted at one time. The concrete is conveyed from the mixers in Insley ½-yd. buckets on flat cars, and the buckets are handled in the yard by a locomotive crane of ordinary capacity, made by the Industrial Works, of Bay City, Mich. Fig. 6 shows the process of concreting one of the blocks—first, the locomotive, then the flat car with the buckets of concrete, then the crane for handling the concrete. From six to ten sets of forms can be filled every day, giving a yardage of 180 to 300 yd. of concrete.

Thirty sets of forms are used. The first cost of the forms averaged 40c. per cu.yd., and the handling averages about 20c. per cu.yd. The mixing and placing of the concrete come to about 60c. per cu.yd.

The steel for the blocks is bent to temples in a bending shed just south of the concreting plant and is carried on flat cars to the block yard, as needed.

At the beginning of the work an attempt was made to fill the forms with a narrow-mouth concrete bucket, but the concrete clogged. It was later found more simple to have boards over the top of the interior forms, leaving only the wall opening, and to dump the concrete from a large bottom-dumping bucket directly on top of the forms, to be shoveled into the walls later.

These forms are allowed to remain for 30 hr., when they are removed to an empty pallet somewhere else in the yard. At the end of 28 days the blocks themselves are taken away and stored in the yard, leaving available other pallets on which the succeeding blocks may be cast. For handling the blocks a 160-ton crane, also made by the Industrial Works, of Bay City, Mich., is in use, and special tongs have been manufactured for the Halifax work. Fig. 7 shows this crane lifting one of the 60-ton blocks. The crane, a regular wrecking crane, is equipped with outriggers on the side and is blocked up from the ground on each side whenever a concrete block is lifted.

The tongs, which are also shown in Fig. 7, consist of a special framework of structural steel, hung from heavy chains with curved grips that fit into openings left for the purpose in the concrete blocks. These openings are arranged on the different-sized blocks in such position that the tongs can always carry the blocks in balance. There is a special pulley arrangement on the tongs, whereby they can be opened and shut. These same tongs are being used on the crane that carries the blocks to be deposited on the harbor bottom.

Power House and Other Details

The power house furnishing the electric power and compressed air for the entire job is located just south of the concrete mixer. The equipment of this house is as follows:

Two 150-hp. locomotive boilers, two 14x14-in. Ames engines belted to two 100-kw. Westinghouse direct-current generators, one 300-hp. Cochrane feed-water heater and one 12x12x18-in. Rand air compressor for furnishing air to granite-dressing tools. The granite-dressing house is just north of the concrete mixer. This stone is brought by barge from a quarry just across the Northwest Arm, on the west shore of the lower harbor. The stone will be used to cap the quay walls.

The harbor contract at Halifax is held by Foley Bros., Welch, Stewart & Fauquier, with J. P. Porter and R. B. Porter in charge and J. W. Roland as Chief Engineer. For the first year and a half C. D. McArthur, now Chief Engineer of the Blaw Steel Construction Co., was Chief Engineer for the contractor, and most of the plant details described here were worked out under his direction. The terminals are being built by the Canadian Government, with James McGregor as Superintending Engineer.

Bone's Retaining-Wall Patent and Its Anticipations

SYNOPSIS—The patent on a cantilever reinforced-concrete retaining wall, held by F. A. Bone, is being vigorously defended by its owner, who is demanding, and obtaining, royalties from many cities, railroads, etc. Decisions against infringers of the Bone patent have been rendered by Federal courts in Ohio and Colorado. A decision against Bone has just been rendered in Indiana. This article describes the patent and recites what appear to be numerous anticipations in foreign literature.

The situation in regard to patents on reinforced-concrete structures is becoming aggravated. Engineers who in good faith have designed and built structures of supposedly standard types or who in the course of regular practice have developed somewhat novel improvements of ordinary details are being confronted with royalty demands from holders of patents purporting to cover the structure in question. In some cases these patents are unknown or obscure, and the demand is apparently a chance shot fired in the hope of bringing down large game. In other cases the patent has been tested in the courts, and the demand is a part of a well-organized campaign to enforce the property rights of the owners of the patent.

In the latter case the engineer can do little but accede to the demands, if he is convinced that the legal validity of the patent has been established; but the history of reinforced-concrete patent suits has been such that most engineers hesitate to accept the rule laid down by court decisions or out-of-court settlements as final, so inadequately have most such suits been defended by the alleged infringers. Furthermore, in many of the patents the engineering evidence of use prior to the patent application is overwhelming, though there is much difficulty in impressing such evidence upon the intelligence of a court not familiar with engineering.

To this last-named class belongs the Bone patent on reinforced-concrete retaining walls. Covering, as it does, a most common engineering structure—a cantilever reinforced-concrete wall, of which literally thousands of miles have been built in the United States—it would prove a gold mine to its owner if its validity were established. Backed by a favorable Court of Appeals decision, the owner, Frank A. Bone, of Cincinnati, Ohio, is now engaged in a systematic campaign for royalties on all walls of this type and is now prosecuting at least half a dozen suits against cities, counties and corporations for infringement. He has also published an advertisement offering 15% of the royalty obtained to anyone informing his office of a wall on which royalty is finally collected. As a result, it is probable that soon every cantilever concrete wall in the country will be under attack, unless court decisions adverse to the patent deter the owner from further activity.

On account of the thoroughness of this campaign and the number of engineers who have been and will be concerned, *Engineering News* has thought it worth while to publish the history of the Bone patent and a list of what appear to be anticipations of it. Of course, the courts must finally decide as to whether these anticipations actually invalidate Bone's patent; but the engineering profession is entitled to know the facts regarding so important a matter.

Bone Patent; Reinforced-Concrete Retaining Wall

The Bone patent is United States patent No. 705,232 and is dated July 29, 1902. The application was filed Apr. 21, 1899, serial number 713,859. It was issued to Frank A. Bone, his application being dated Apr. 18, 1899. The one drawing accompanying the patent is reproduced in Figs. 1 to 3, and the main paragraphs of the application and of the claim are as follows:

My invention relates to improvements in retaining walls for abutments of bridges, sea walls, banks of streams, embankments, cuts, dams, dry docks and such places as it is desired to retain earth or other matter permanently in place with its face at an angle nearer vertical than it would naturally repose when exposed to the action of the elements or gravity.

The said invention consists principally of introducing into masonry of concrete, stone or brick a framework of steel or iron in such a way that the whole wall is so much strengthened thereby that the volume of the masonry may be greatly reduced, and yet the height, base and strength against overturning, bulging or settling will still be ample.

The object (stability with reduced volume) is further accomplished by the peculiar shape of the cross-section of the wall allowable.

The construction is shown in the accompanying drawings, in which Fig. 1 represents a cross-sectional view of my invention. Fig. 2 represents a side view of one of the metal bents and connections, turned one-fourth around from its position in Fig. 1. Fig. 3 represents a cross-sectional view of the metal on line y z, Fig. 2.

In the drawings similar letters refer to corresponding parts throughout the several views.

In the form of wall shown the base extends somewhat more to the rear than to the front of the vertical part of the wall; but this may be varied to suit the circumstances—that is, the base with proper proportions of metal may be extended almost wholly to the rear or to the front of the vertical part without danger of its breaking off—the form shown in the drawings being what might be called an inverted T, while those suggested would be in the form of an L or reversed L. A represents the masonry; B represents the material retained; B′, the earth on which the wall rests.

It is evident that a wall of the shapes mentioned is not so liable to be overturned from the pressure of material behind it as would a wall of the same height and area of section, but having a rectangular trapezoidal or triangular-shaped section, for it is impossible to obtain as much base and height with a given amount of masonry with these last-named shapes as it is with those described above. This wall, also having more base and less weight, will rest more securely on a soft or yielding foundation. The weight of the material resting on the heel acts, however, to cause said heel to press on the earth below and thus cause friction to prevent the whole wall from sliding outward.

The retaining wall is preferably composed of cement-concrete masonry with skeleton or framework of steel or iron located at the back parts, where the greatest tensile strain comes, and of other strengthening rods of metal located at the lower part of the toe. Brick or stone masonry may, however, be used in place of concrete, or a concrete backing may be used with a brick or stone face.

[Here appear paragraphs giving a detailed description of the reinforcement.—Editor.]

In a soft or yielding foundation a row of piles n are placed along under the toe of the wall. With the piles driven to a firm foundation or with the toe of the wall resting on a firm foundation the wall in order to be overturned by a pressure in the rear would have to turn on the toe of the wall with a pivot near where the pile n is located in the drawings. In this case it is evident that the body of the wall, together with

the earth or other material resting on the back of the base, will have to be lifted upward. Thus it can be seen that the shape of the wall as described is such that the tendency to be overturned by pressure on the back is overcome with a minimum amount of material in the wall; but this reduced amount of material in masonry must be reinforced by the introduction of a stronger material than masonry where the greatest tensile stress comes.

With the earth pressing outward on the vertical part of the wall and downward on the heel or back part of the base it is evident that the internal stresses created within the wall will act to create tension in the rear and compression in the front, or the pressure outward results in an upward pull at the back and a downward push on the front part of the vertical part of the wall. Now ordinary good masonry of concrete, brick, or stone is as good a material as we could desire to resist the compressive strains on the front, but with a wall as described previously and shown in the proportions of the drawings the best of masonry would be deficient in

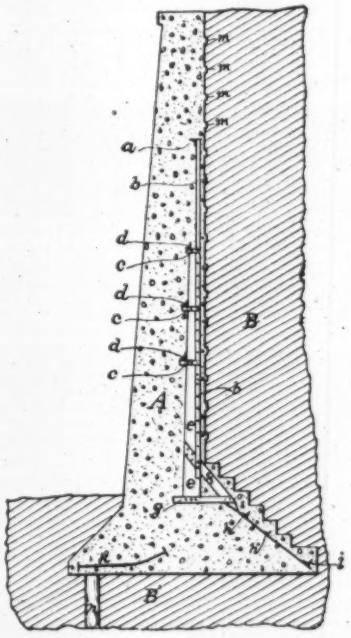


Fig. 1.

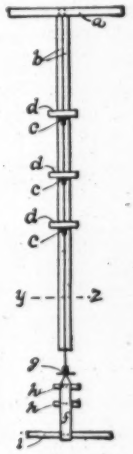


Fig. 2.

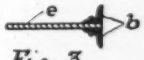


Fig. 3.

FIGS. 1 TO 3. FACSIMILE OF DRAWING ACCOMPANYING PATENT NO. 705,732, ISSUED TO FRANK A. BONE

strength in the back part and would be liable to crack open and the body of the wall break off and fall outward. I therefore place my metal in the extreme back part of the wall, where the tension is greatest and where the metal will have the greatest effect, being farthest from the neutral axis of the wall. The upright of metal is, however, made stiff enough to resist any tendency of the wall to bulge that might occur in making a wall as thin as the one described.

The T-shaped angle plate b, a cross-section of which is shown in Fig. 3, is secured to the rear side of the upright member e and is used on account of its stiffness and rigidity and because the greatest mass of metal is placed in the extreme back part of the wall, where the greatest strain comes and where it is of the most service.

The back part of the base or the heel of the wall under the forces mentioned would be subject to a tension in the upper portion. I therefore put in the tie-brace f at this point.

In building my retaining wall the metal skeleton is first erected in place, each intersection being fastened together with a sufficient number of rivets or bolts. The masonry is then built around the frame, completely covering it and protecting it from the elements. The projections m m m, etc., on the back part of the wall run horizontally along the length of the wall and are for the purpose of increasing the sliding friction between the wall and the material retained, and thus assisting to keep said wall in its normal position. The retained material catches on the projections and creates considerable downward pressure on the back part of the wall as a resultant of the weight of the retained material. When the material retained is in a liquid state, I claim no advantage to be derived from the projections mentioned.

I am aware that retaining walls have been constructed of concrete and steel, but none to my knowledge have been supported on their own base as mine, nor have any of them entirely inclosed the steel within the concrete, nor have any of them used the weight of the material retained as a force to retain itself.

Having thus described my invention, what I claim as new and desire to secure by letters patent, is:

1. The combination with a retaining wall having a heel, of a metal structure embedded vertically in said wall and obliquely in said heel, so that the weight of the retained material upon the heel of the metal structure will operate to retain the wall in vertical position.

2. In retaining walls the combination with a wall of masonry, a series of tapering upright bents or members of metal braces embedded within the interior of said masonry at the back part, only, of the vertical portion and continuing down along the upper part of the heel of said wall to the back part thereof, substantially as and for the purposes set forth.

3. The combination with a retaining wall having an inclined heel, of a metal structure embedded within said wall and heel, consisting of upright bents at the back part of the vertical wall and continuing down along the upper part of the heel of said wall to the back part thereof, in such a manner that the weight of the retained material upon the heel of the metal structure will operate to maintain the wall in vertical position.

4. A retaining wall having a heel (strengthened by metal within and connected with braces in the wall by suitable brace rods) extending to the rear of the base, upon which the retained material rests and acts by gravity to keep said wall in its normal position, substantially as described.

5. The combination with a retaining wall having a heel, of a metal structure embedded within said wall and heel, consisting of upright bents at the back part of the vertical wall and continuing down along the upper part of the heel of said wall to the back part thereof in an inclined direction, whereby the weight of the retained material upon the heel of the metal structure will operate to maintain the wall in a vertical position, as set forth.

6. In a retaining wall a series of tapering bents embodied in said wall composed of the upright member e, base g and tie-brace f in combination with the masonry substantially as described.

7. In a retaining wall a series of bents of metal in the back part of said wall extending down along the upper part of the heel, and a series of strengthening rods k provided with enlarged ends and embedded in the lower part of the toe of said wall, in combination with the masonry of said wall, substantially as described.

8. In retaining walls a series of bents composed of the tapering upright member e having flanges b, the tie-brace f and base brackets g in combination with the masonry, substantially as and for the purpose set forth.

9. In retaining walls a series of upright bents tapering in size and strength from the base to the top, angle rods i and braces f between said rods and said bents, in combination with the masonry, substantially as described and for the purpose set forth.

10. In a retaining wall the combination with the masonry thereof, of upright bents having their lower ends suitably anchored and braced and provided with suitable angle plates and brackets, said angle plates and brackets being adapted to be so held in said masonry that any vertical or lateral slipping or movement of said plates or brackets independent of the movement of the upright bents and the wall will be prevented.

11. In retaining walls the anchor bars d attached to the bracket c in combination with the upright member e and wall A substantially as and for the purposes set forth.

12. In a retaining wall the beam i attached to the brace f in combination with the masonry A, substantially as and for the purposes set forth.

13. In a retaining wall the combination of the metal framework consisting of the longitudinal bars a and i and a series of tapering upright bents and braces between said bents and the bars i, with a wall of masonry inclosing said framework substantially as and for the purposes set forth.

14. In retaining walls the brackets h attached to the brace f in combination with the masonry A for the purpose set forth.

15. In a retaining wall the base g in combination with the upright member e, tie f and masonry A, substantially as described.

16. The combination with a retaining wall having a heel and a toe at opposite sides thereof, said toe having an independent metal structure embedded therein, of a metal structure embedded within said wall and heel, said structure consisting of upright bents at the back part of the vertical wall and continuing down along the upper part of the heel of said wall to the back part thereof, so that the weight of the retained material upon the heel of the metal structure will operate to maintain the wall in a vertical position.

17. The combination with a retaining wall having an inclined heel and a toe at opposite sides thereof of a metal structure embedded within said wall and heel, said structure consisting of upright bents at the back part of the vertical wall and continuing down along the upper part of the heel of said wall to the back part thereof, whereby by reason of the toe and the heel the weight of the retained material upon the heel of the metal structure will operate to maintain the wall in a vertical position.

History of Patent and Suits

It is understood that Mr. Bone claims to have conceived the wall some time before the first application for his patent (1898). There was the usual discussion between the Patent Office examiner and the inventor, which is on record in the department, but the patents cited by the examiner do not appear to have been important.

So far as can now be discovered, there was no publication about Bone's wall until the latter part of 1901 or the first part of 1902. In *Engineering News*, Mar. 27, 1902, p. 242, there is a brief article describing the new type of wall invented by Mr. Bone, accompanied by a drawing which is practically a copy of the Patent Office drawing.

After the publication of this article there were two letters in *Engineering News*, one by David A. Molitor, now of Toronto, Ont., the other by J. Lehman, of Copenhagen, Denmark. Mr. Molitor claimed that he had anticipated Bone's invention in a paper he read before the Cornell Society of Civil Engineers in June, 1899, published in the "Journal" of the Association of Engineering Societies, Vol. 24, No. 1. Mr. Lehman claimed that he had built walls of precisely the same type in Denmark, as early as March, 1898, and that his invention was protected by patents in Denmark. The wall shown in Fig. 6 is the Lehman wall, Danish patent No. 3,316, applied for, curiously enough, on the same day as the Bone patent—Apr. 21, 1899.

Subsequent to 1902 a number of articles appeared on L-shaped retaining walls, some of them by Mr. Bone and by Evan P. Bone; but little or no attempt was made to demand royalties under the patent. About this time the American literature of reinforced concrete began noticeably to increase; and from then on, the L-shaped retaining wall has become a fixture in the literature of the subject. In none of these books, however, has any definite statement been made that the L-shaped wall is patented.

The first suits brought by Mr. Bone were commenced in 1912 and 1913. Since that time at least two decisions have been rendered in his favor, the most important of which is that against the City of Akron, which was decided in the Federal District Court for the northern district of Ohio, eastern division, in 1914, and affirmed by the United States Circuit Court of Appeals, sixth circuit, before Judges Warrington, Knappen and Dennison, on Mar. 2, 1915. Judge Dennison wrote the decision. This decision has been used by Mr. Bone as his main support in other suits that he has brought and also as a potent argument in favor of his patent in any controversy with alleged infringers.

Another famous suit was that against the Chicago, Burlington & Quincy R.R., which was settled out of court. This suit against the railroad was for $400,000, and on advice of counsel the railroad compromised the case with Mr. Bone for $18,000. The photograph of this large check has been successfully used by Mr. Bone in his propaganda. The third case, in Denver, was settled in Mr. Bone's favor mainly on the basis of the Ohio suit, the defendant not having any new evidence to present.

A fourth case, tried in Indianapolis before Judge Anderson, was decided against Mr. Bone on Jan. 26, 1916 (*Engineering News* of Feb. 3, p. 246).

In this last case, there were brought to the attention of the court, for the first time, a number of foreign publications showing what were claimed to be direct anticipations of the Bone patent. As mentioned last week, Judge Anderson in his opinion said he was convinced that in the Akron case the court did not have all the facts before it and he believed the prior art disclosed walls embodying the essentials of the Bone invention. His main point was that under these circumstances the patent could not be construed broadly and, if it were valid, the claims must

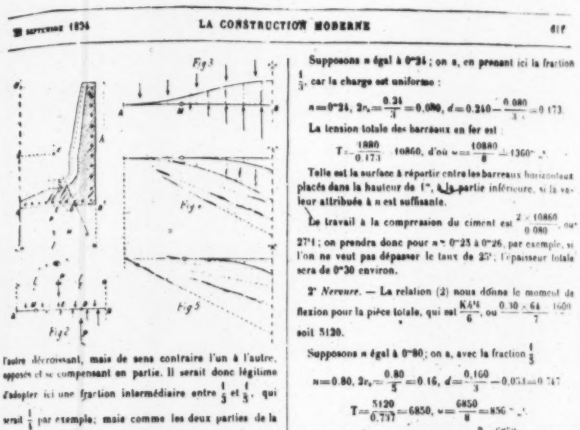


FIG. 4. FACSIMILE OF PAGE 611 OF "LA CONSTRUCTION MODERNE," SEPT. 22, 1894, SHOWING PLANAT'S WALL

be restricted to the particular form of reinforcement shown by Bone.

It is the intention of this article not to discuss the legal complications of the Bone patent; that is entirely beyond the field of this journal. What it is purposed to do is to present in as brief form as possible the state of the art of designing reinforced-concrete retaining

walls up to the time that Mr. Bone made application for his patent; to show, in so far as is possible, the engineering aspects of the retaining-wall patent situation.

It is quite possible that there may be legal conditions which make the Bone patent valid. It is quite possible that Mr. Bone can collect millions of dollars' royalty on the hundreds of millions of dollars' worth of L-shaped reinforced-concrete walls that have been built in the last decade. But it is due to the members of the engineering profession who may have to see this royalty added to the cost of their structures that they have presented to them a full statement of what the profession knew concerning such structures at the time the patent was issued.

Further, what is purposed to be shown here is that the minds of engineers during the last decade of the nineteenth century were turning toward the L-shaped retaining wall, coincidentally with the developments of reinforced concrete. Contrary to current opinion, reinforced-concrete theory was pretty fairly developed in Europe in the middle '90's, and there were at that time a number of highly scientific contractors at work there on the construction of reinforced-concrete structures whose details would not be out of place in the best modern practice. These men were building L-shaped reinforced-concrete retaining walls. Unfortunately for those who may have to pay Mr. Bone royalties, they were not writing much about them. But in one or two well-circulated publications there may be found descriptions of walls, which, to the engineering mind at least, clearly establish the fact that at that time—at least three years before Mr. Bone's application—the reinforced-concrete wall was placed before the public in such clear fashion that anyone could have designed and built such a wall, had he been so minded.

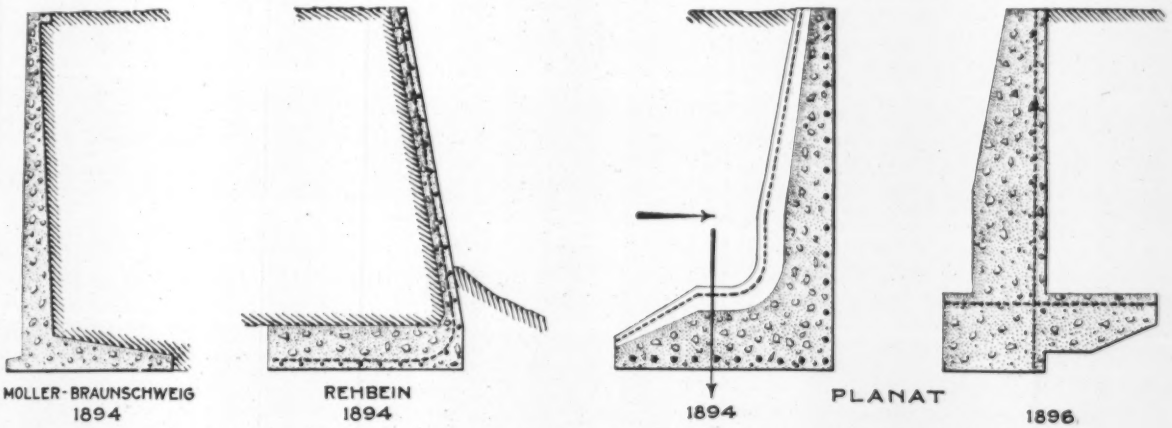


FIG. 5. FOUR EARLY CANTILEVER RETAINING WALLS OF REINFORCED CONCRETE
Dates shown are dates of publication

Wall Designed by P. Planat

The first reference goes back to December, 1893. At that time P. Planat was editor of the French publication *La Construction Moderne*. In December, 1893, he started in his journal a serial article entitled "La Théorie de Ciment Armé" (The Theory of Reinforced Concrete). This article may be commended to those American engineers who think reinforced concrete is a product of the present century. The discussion is wonderfully complete. The theory has its peculiarities, but it is remarkably sound in its fundamentals. An engineer entirely unacquainted with the art of reinforced-concrete design, but skilled in the theory of mechanics, could readily establish himself as a reinforced-concrete expert on the basis of M. Planat's treatment.

Every week this course appeared, and on Sept. 22, 1894, the current article was entitled "Retaining Walls" (Fig. 4). The first paragraph of this article, freely translated, is as follows:

> We have no knowledge that reinforced concrete has been as yet applied to the construction of retaining walls. It can, however, be used there most advantageously. Masonry walls must have a very great volume, because they resist only by their mass. A rigid and at the same time an elastic structure can necessarily have very much reduced dimensions and can be used with a very sensible economy. Iron has these advantages, but the bare metal so employed will not in contact with earth last as long as iron buried in cement, for there it has the priceless property of remaining indefinitely without any alteration. We will now indicate how one is able to establish the calculations for a retaining wall.

Following this, M. Planat, with the aid of a drawing, a copy of which is shown in Fig. 5, proceeded to analyze the stresses and strains in the L-shaped retaining wall shown there. This wall, it will be seen, is a cantilever retaining wall reinforced on its outer face, a precise original of the modern reinforced-concrete cantilever wall, except for the fact that the rods in the drawing are shown running parallel to the length of the wall, as well as vertically in the face and horizontally in the inclined base. The text of the analysis would indicate that M. Planat intended that such rational reinforcement be used, but the indications are not sufficient to insure that theory beyond dispute.

Two years later, however, in the issue of *La Construction Moderne* for Sept. 26, 1896, there appeared another article on reinforced-concrete retaining walls. In it appeared the drawing shown also in Fig. 5. This article was in response to the query of a correspondent who submitted a gravity masonry retaining wall and asked the editor whether it would not be possible to design such a wall according to his well-known principles. The editor replied that such a design would be quite simple, and he submitted the drawing shown, together with analysis of that wall for the dimensions demanded by the massive wall shown by the correspondent.

The important feature of this second Planat wall is the reinforcement shown running vertically along the inner face of the wall and along the upper plane of the base. In this connection the following free translation of

some paragraphs in the 1896 wall discussion is of interest:

> These calculations presuppose that the tying together of the vertical wall and the horizontal base at their juncture has been effectively performed. This tying together requires special precautions. The reinforcing bars at their point of juncture have a tendency to pull out of the concrete. It is necessary that the ends of the bars in the section where they are tied together be extended sufficiently into the mass of concrete or be maintained by some other method. If the bars should be extended so that the adherence to the concrete governs, it is necessary for the extension to be a considerable length. It is necessary, then, that the base extend a considerable distance behind the wall and that it be lowered for quite a distance below the wall.
>
> One can reduce the extensions in great measure, if one takes care to join together the vertical bars and the horizontal bars at their juncture. In this manner the tension of a bar extends not only along its own length, but is transmitted into the bar which is perpendicular to it, of which the great length permits it to offer a great resistance to the transverse tearing-out effect.
>
> In this sort of construction it is evidently the tying together which should take all the attention of the constructor. One can, moreover, consolidate the wall by means of some bars placed at 45° in the dangerous angle and buried in the two walls perpendicular to each other.

While the Planat walls are the most complete analysis of the prior art, there are one or two others which should be resurrected from the libraries.

Rehbein and Moller-Braunschweig Walls

Reproductions of the drawings illustrating the Rehbein and Moller-Braunschweig walls are shown in Fig. 5. The Rehbein wall appears in a book by F. Rehbein, entitled "Ausgewählte Monier- und Beton-Bauwerke," published in Berlin in 1894. The book illustrates a number of reinforced-concrete structures built under the Monier system, among which is the bridge at Greifenhagen, built in 1891, which contains the wall shown in Fig. 5. There is no other description of this wall.

In the *Deutsche Bauzeitung*, toward the end of 1894, there appeared a serial article by Prof. M. Moller-Braunschweig, entitled "Empirische Untersuchungen im Bau-Ingenieurfach, insebesondere an Beton-Eisenkonstructionen ausgeführte Bruch-Belastungen." In the issue of Dec. 15, 1894, p. 621, appears among other concrete retaining walls under investigation the section shown in Fig. 5. No reinforcement is shown in the drawing, but the article plainly indicates that reinforced concrete, as understood today, was the material of the wall.

Cantilever Walls in General

It will be noted that all of these walls are pure cantilever walls, with no counterforts or buttresses. The theory of a cantilever wall with buttresses is of much wider circulation. Many references to such walls might be cited. This record will confine itself to the following:

On Apr. 23, 1897, François Hennebique, the well-known French reinforced-concrete expert, made application to the Government of Great Britain for a patent entitled "Improvement in Piles, Quays and Retaining Walls or Structures." This patent was issued under No. 10,203 (1897) on July 3, 1897. It is mainly on cast concrete sheetpiles, but in the drawings which accompany the patent are shown retaining walls of accepted modern type. While patents were not claimed on these walls specifically, they are shown as a part of an ordinary construction, demonstrating that at that time Hennebique did not consider such walls of sufficient novelty to be patented (Fig. 7).

In the September, 1898, issue of *Le Ciment*, a journal published in Paris, France, is an article entitled "Sur un Nouveau Mode de Construction en Béton Armé des Murs de Reservoirs de Grande Capacité" (A New Method of Reinforced-Concrete Construction for Walls of Reservoirs of Large Capacity), by Lieutenant-Colonel Le Besnerais, of the engineering corps of the French army. This article gives a full theoretical discussion of reinforced-concrete cantilever walls. The author does not say anywhere that such walls have been built, but suggests them for reservoir use with fill behind and water in front. In the drawings the walls show a toe as well as a heel. Among other things, the author says: "By the use of a metallic skeleton conveniently disposed, the tensile resistance of concrete already greater than that of ordinary masonry is considerably increased. It is the same way with its resistance to flexure and to shear."

As further evidence that cantilever walls were built in Europe before the issuance of the Bone patent, reference may be made to one wall of great prominence.

The wall is at a pier at Southampton, England, and a description is to be found in the "Annales des Travaux Publics de Belgique" for June, 1899, pp. 518-520, in an article entitled "Béton Armé et les Applications," by Paul Christophe. This series of articles was afterward revised and published in book form in 1902. Part of the wall is shown in Fig. 8. The article says that the wall was built in 1898 for the London & Southwestern Ry. It goes on to say that the system of construction which is described here for quay walls can also be used for bridge abutments and that an application of this kind had been recently made for the construction of the abutment of a bascule bridge at Portsmouth.

The main United States patent which might at all resemble the Bone patent is No. 629,487, issued to Charles F. Stowell and Andrew C. Cunningham, of Albany, N. Y. This patent was filed Mar. 25, 1897, and was granted July 25, 1899. This wall is a cantilever wall made up of continuous metal plates in a vertical and horizontal planes, fastened together at their junction and covered on each side with concrete or mortar. The important fact about it is that it utilizes the weight of the material to increase the stability of the wall. In fact, part of the patent reads, "The weight of the mass of material superincumbent above the foundation keeps the latter in place and prevents overturning." The wall, of course, was not of reinforced concrete.

What Does the Bone Patent Cover?

It is of course impossible for anyone to lay down, beyond danger of judicial contradiction, just what a patent covers; but to the engineering mind the description, drawing and claims in the Bone patent, stripped of legal and patent-office requirements, present the following structural elements:

1. A cantilever retaining wall; that is, a wall in which the weight of earth on an outstanding base integral with the vertical part of the wall acts to resist the thrust of the same earth, tending to overturn the wall. Such a wall may also have a base extending away from the earth-fill, known as a toe, which increases stability by virtue of the enlargement of the base.

2. A steel framework embedded in the concrete or masonry of the wall in such a location as best meets the stress conditions set up by the load. This stipulation requires that the steel be placed in that part of the section where tension exists and that there be a definite in-

tegral connection between the vertical and horizontal steel.

3. A definite form of steel framework described throughout as a "bent," the distinguishing features of which seem to be a structural unity as distinguished from articulated rods, a taper in size and strength from base to top and provision for angle plates and brackets to prevent slipping of the plates in the masonry.

Let us see how these provisions are anticipated by the publications noted in this article.

1. The cantilever-wall principle was decidedly not new in 1898. Every one of the walls cited, from Planat and

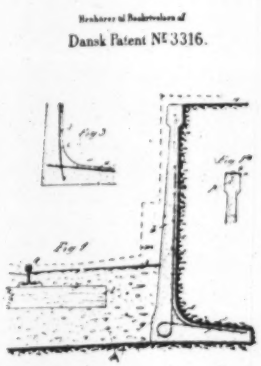


FIG. 6. LEHMAN DANISH PATENT, 1899

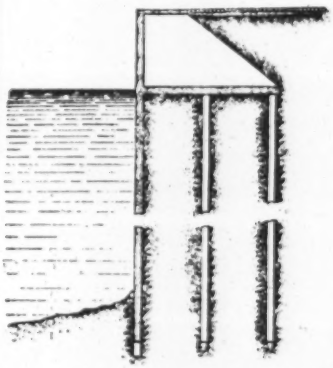


FIG. 7. HENNEBIQUE ENGLISH PATENT. NO. 10,203, 1897

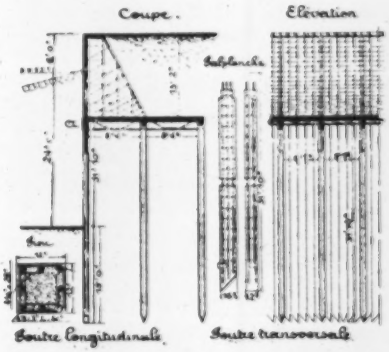


FIG. 8. DETAILS OF WALL AT SOUTHAMPTON, ENGLAND, 1898

Rehbein in 1894 to Stowell and Cunningham in 1897, shows a definite appreciation of the advantages to be gained by extending the base of the wall to carry part of the earth-fill.

2. Reinforcement. In so far as stone or brick masonry is concerned, there is no available anticipation of the Bone patent; in fact, there is none on record as being built by Bone or by anyone else. In so far as concrete is concerned, the Bone design brought only an application of the general principles of reinforced concrete to retaining walls. Reinforced concrete obviously was not new in 1898. The placing of steel to care for readily figurable stresses was of much earlier date. Planat, with only a theoretical knowledge of reinforced concrete, was able to apply that knowledge to the design of a cantilever retaining wall. His 1894 wall is possibly not as detailed in its reinforced-concrete design as the later wall; but his 1896 wall is complete in that it shows steel in the base of the wall and the top of the base, the necessity for tying the two sets of steel together and the advisability of using an inclined brace between them.

3. Type of reinforcement. Records do not show any other such type of reinforcement as that shown in the Bone patent. It was, in fact, exceptionally clumsy even in 1898 and indicated a lack of appreciation of the elements of reinforced-concrete design. Compare, for instance, the Bone reinforcement of 1902 with that of the 1898 Southampton quay wall, shown in Fig. 8. From an engineering standpoint the Bone reinforcement is novel; from a legal, it probably would hold. It is quite improbable, however, that anyone in these days would care to imitate it in a structure.

Attention may be called to the two more important legal decisions on the Bone patent. The first one, rendered by the United States Circuit Court of Appeals on Mar. 2, 1915, upheld the lower-court decision in favor of the Bone patent against the City of Akron. In that, Judge Dennison said:

> If the prior art had shown a structure intended for a retaining wall and having a heel such as the weight of the earth thereon would tend to keep the wall erect, it might be difficult to find invention in merely adding the form of reinforcement most suitable to create the desired tensile strength; but we find no such earlier structures.

In this suit the Planat and Moller walls were not brought in as evidence. So far as one can tell, these walls do specifically show the prior art that Judge Dennison remarked did not exist. In the suit against the County of Marion, brought before Judge Anderson in Indianapolis in January, 1916, the Planat, Rehbein and other earlier walls were brought in evidence, and with these walls before him Judge Anderson stated that evidently the Ohio court had not full evidence presented to it. He declared that the patent applied only to its specific type of reinforcement.

Engineers' Attitude Toward Bone Patent

Engineers quite generally have assumed that the Bone patent is invalid, probably because the cantilever reinforced-concrete retaining wall is now common knowledge of the profession and is included in all of the literature. It must be remembered, however, that up to the week of January 21 of this year Mr. Bone had a well-authenticated legal right to his patent; for in the one suit which had been brought to a higher court, the patent had been definitely and decisively upheld. He was, therefore, under the law of the United States quite justified in prosecuting any infringement, although the methods which he took for such prosecution might be distasteful to the engineering profession.

The average engineer is apt to take a snap view of the legality of the Bone patent. On account of the general acceptance of the design and its wide publication in literature he has come to think that such a wall has always been in use. The fact is that Bone, so far as records show, was the earliest American adapter of a European idea. American practice does not show anything precisely like it before him, with the exception of the one American patent mentioned in this article, in which there are certain vital differences. But the fact remains that Mr. Bone could have read the European technical papers at any time from 1893 to 1897, when he claims to have conceived the idea, or he could have visited Europe in 1897 and, either in European publications or in actual steel and concrete, have seen structures answering to the general type of the wall which he claims to have invented.

Bascule Bridge of Short Span

The new bascule bridge over the Ecorse River, in Michigan, with a clear channel width of 60 ft., is an interesting example of drawbridges of the smaller class.

The development of an industrial district south of Detroit, made it desirable to open up the Ecorse River for navigable connection with the Detroit River, dredging the old channel to the required dimensions. The mouth of the stream was closed by two pile trestles, one for Jefferson Ave. (Ecorse township) and the other for an interurban electric line. It was decided to replace these by a single bridge with a drawspan.

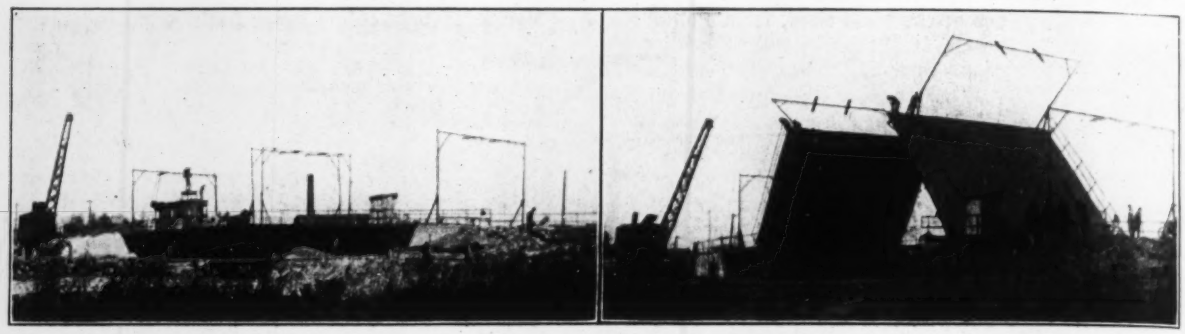


FIG. 1. ROLLING-LIFT BASCULE BRIDGE OF 60-FT. SPAN OVER THE ECORSE RIVER, NEAR DETROIT, MICH.

The bridge is of the Scherzer rolling-lift bascule type, with a double-leaf deck-girder drawspan 67 ft. c. to c. of bearings (closed) and a short girder approach span at each end. It has a 32-ft. roadway (with double-track car line) and two 5-ft. sidewalks. Fig. 1 shows the bridge in its closed and its open positions.

The substructure consists of two abutments and two pairs of pedestal piers, founded on timber piles driven into the clay soil. Steel sheetpiling was driven for each pier and abutment, that for the piers extending 4 ft. below the bottom of the new channel. The coffer-dams were pumped out and foundation piles driven. Then the material was excavated to 4 ft. below the pile heads, and the concrete was placed. The sheeting was afterward removed. For each pedestal pier there are 14 piles 40 ft. long and 14 piles 55 ft. long.

Each pier is 11½x15 ft. up to mean water level, above which is a tapering pedestal or bridge seat, 4 ft. high and 4x10 ft. on top. In this is embedded a plate girder, with its top chord flush with the surface of the concrete and carrying the heavy track plate on which the heel of the bridge girder rolls. The front end of the track girder (which forms the main bearing when the bridge is closed) rests on an I-beam grillage, and both ends are secured to the body of the pier by long anchor bolts. The concrete is proportioned 1 : 3 : 5, using washed gravel of ½- to 2-in. size.

The drawspan is of the double-leaf type, as this arrangement requires considerably less weight of material than a single-leaf bridge. For each leaf there are two main plate girders about 45 ft. long, including the counterweighted tail ends. Their bottom chords are curved, which improves the appearance of the structure. They are spaced 30½ ft. c. to c., and between them are framed plate-girder floor beams, while on the outside are cantilever brackets for the sidewalks. Diagonal bracing is placed in the plane of the tops of the floor beams, the laterals being riveted to the floor joints. The construction is shown in Figs. 2 and 3.

The rolling heel of each girder consists of segmental steel castings having straight backs riveted to 8x6-in. angles. This is more satisfactory than the earlier construction with steel plates shaped to the curve of the heel. In the castings are pockets which engage with the double row of teeth on the track plate. These teeth, or lugs, prevent longitudinal creeping and hold the leaves in line during their travel.

The counterweights consist of steel boxes forming the tail ends of the girders and filled with concrete, which

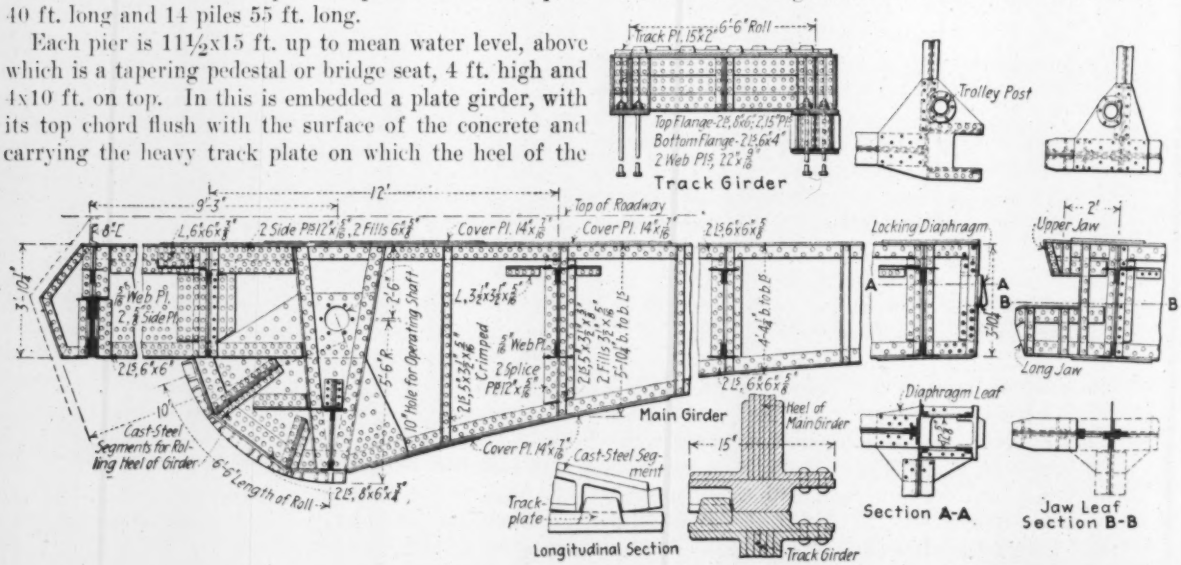


FIG. 2. MAIN GIRDERS, WITH CURVED HEELS AND TRACK PLATES, FOR ROLLING-LIFT BASCULE BRIDGE

is reinforced by bars on the outer face to prevent cracking. This concrete weighs about 146 lb. per cu.ft., and there are 74 tons of concrete on each leaf. The weight includes concrete blocks to adjust the balancing of the bridge. They are 12x12-in., 6 in. thick, placed in pockets in the monolithic concrete.

The bridge is designed under Cooper's specifications for highway and electric-railway bridges, 1909. The assumed live loads are as follows: For the trusses, 2,000 lb. per lin.ft. on each track, and 80 lb. per sq.ft. of side-

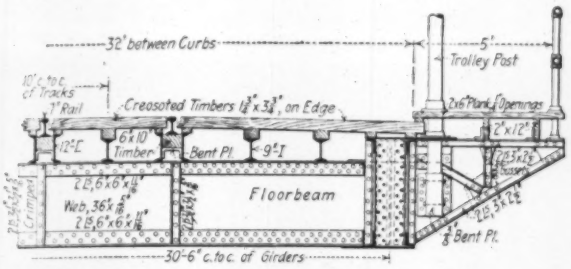


FIG. 3. CROSS-SECTION OF BRIDGE, SHOWING FLOOR CONSTRUCTION

walk; for the floor, a 50-ton car on each track or a 12-ton truck on any part of the roadway.

There are 11 lines of stringers for the roadway, 7 of these being 10-in. I-beams to carry the paving. The others are pairs of 12-in. channels under the rails of the car tracks. Wood sleepers on the stringers form spiking pieces for a deck of mill-construction type having 1¾x3¾-in. surfaced maple strips laid on edge across the bridge. This deck is covered with a layer of pitch and limestone screenings and makes a light but rigid floor. The sidewalks have 2x6-in. planks, laid flat and ¼ in. apart, upon wooden joists.

The car tracks have 7-in. 90-lb. T-rails. These rest upon 8x6-in. longitudinal timbers, which are carried by saddles of bent plates riveted between the pairs of channels. The saddles are 6 in. long, spaced 3 ft. c. to c. Cement grout filling against the webs of the rails forms a backing for the deck. The rails of one leaf project far enough to rest upon seats on the other leaf, so that there is no open joint in the track.

The trolley posts are of iron pipe, placed just outside the roadway wheel guard and resting in sockets on the sides of the cantilever brackets. Each pair of posts is connected by a 4-in. pipe across the roadway, with a T-bar kneebrace at each end.

A special feature of the design is the absence of moving end locks, with their operating mechanism. This is effected by having projected diaphragms on one leaf, which engage with jaws on the opposite leaf (Fig. 2).

With the combined longitudinal and lowering movements of the leaves in closing, the bridge when fully closed has the diaphragm of one leaf engaged between the upper and lower jaws of the other leaf, so that the ends of the leaves cannot move independently as loads pass across them. Nor can the leaves be lowered beyond their proper position, so that no tail locks or anchor locks are required. As the leaves reach their raised position, the tail ends engage bumpers on the rear of the piers. In full open position the roadway is at an angle of 70° with the horizontal.

The operating machinery, Fig. 4, is carried entirely within each leaf and is of very simple character. A train of gearing ends in pinions outside of the main girders, and the pinion shafts are in the center of rotation of the leaf, so that they have simply a horizontal travel as the leaf rolls back or forward. The pinions engage horizontal racks on the inside of the fixed girders over the pier.

Each leaf will have a 7½-hp. motor, with an automatic brake which is released by the first movement of the controller handle before current is delivered to the motor. The machinery will be controlled from a cabin at one end of the bridge. There is no necessity for frequent operation of the bridge at present, so that the motors have not yet been installed.

The hand-operating gear has crank handles on a shaft carrying a pinion which can be shifted along the shaft to engage the first spur wheel of the main gear train. The operators stand in a "squirrel cage" operating pit, the curved floor of which (below the bridge deck) rolls beneath them as the bridge moves. A hand-operated brake is provided to hold the bridge in any position when electric current is not used. The bridge can be opened by hand in about five minutes.

The operating pinions and racks have a pitch of 2¼ in. The gear train is as follows: (A) Motor pinion, 4 in. in diam., 16 teeth; (B) spur wheel, 27 in., 108; (C) pinion, 5 in., 15; (D) spur wheel, 30 in., 90; (E) pinion, 7 in., 14; (F) spur wheel, 38.5 in., 77; (G) rack pinion, 10.03 in., 14; (H) hand pinion to (B), 10 in., 40 teeth.

The bridge was designed by the Scherzer Rolling-Lift Bridge Co., of Chicago, acting as engineer for the Ecorse Township Commissioners. The substructure was built by M. Rabbitt & Sons, and the superstructure by the Toledo

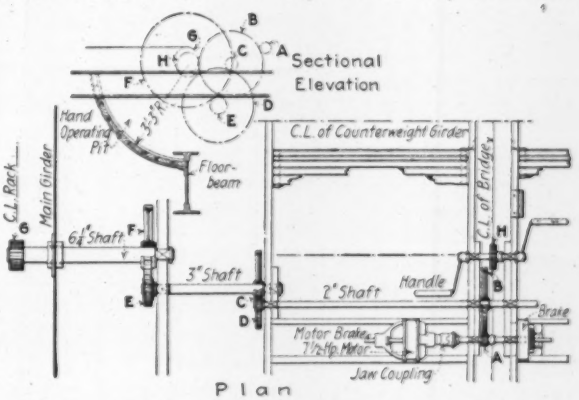


FIG. 4. OPERATING MACHINERY OF THE ECORSE RIVER BASCULE BRIDGE

Crane and Bridge Co., both of Toledo, Ohio. It was erected on falsework in the closed position, the river not being navigable at that time.

Work on the foundations was commenced May 12, and the substructure completed Aug. 20. Erection of steel was begun Aug. 11, and the bridge completed on Oct. 10, 1915. The total weight of the superstructure, including operating machinery, is about 120 tons. The total cost was $29,450, of which $19,000 was for the superstructure and operating machinery.

The Patrol System of Road Repair in Essex County, New Jersey, now in its third year, is said by Frederick A. Reimer, County Engineer, in a report issued by him at the beginning of the present year, to be giving excellent satisfaction. Mr. Reimer says that it is very evident that the patrolmen have advanced considerably in the knowledge of their duties and are today making repairs in a most efficient manner and at a much less cost than under the old system.

Activated-Sludge Experiments at Milwaukee, Wis.*

By T. Chalkley Hatton†

SYNOPSIS—A number of activated-sludge tanks of different designs have been operated by various methods with such promising results that a start has been made on the construction of a 100,000,000-gal. plant.

I-Plant Used; Diffuser Tests

Experiments on the activated-sludge process of treating sewage have been carried on by the City of Milwaukee since the beginning of 1915. They have been greater than those carried on elsewhere in the world. The experiments have been conducted under the immediate direction of William R. Copeland, chief chemist to the Sewerage Commission, who has spent 20 years of his life continuously upon experimental work connected with water and sewage purification. He has been ably assisted by a staff of two chemists, two bacteriologists, three engineers and several unskilled men.

The plant has been equipped with the best apparatus available to determine the volume of sewage treated and the volume and rate of air applied. It has been run continuously night and day, always under skilled supervision.

Early in March, 1915, laboratory experiments were begun with two 1¼-in. by 6-ft. glass tubes. These early experiments showed: (1) That with a small volume of air per minute almost any degree of nitrification required could be obtained if the air was applied a sufficient length of time. Thirty parts of nitrates in the effluent after 42 hr. aëration were common. (2) That there appeared to be little difference in the character of effluent obtained by the same volume of air, whether it was applied through an open jet or a filtros plate.

Fill-and-Draw Tanks and Method of Operation

The second series of experiments was conducted in a tank 32 ft. long, 10½ ft. wide and 10 ft. deep, having a working capacity of 22,200 gal. In the bottom of this tank there were 51 filtros plates, 12 in. square and 1½ in. thick. The ratio of plate surface to tank surface was about 1 to 6.6. The coarse-screened sewage was discharged directly from an orifice box at one end of the tank, and the effluent was drawn off by means of a floating circular weir.

This tank was put in operation on May 6, 1915, and continuously operated until Nov. 24, 1915. Numerous variations were made in the cycles, volume and rate of air, but under normal conditions the cycle was as follows: Filling, 1 hr.; aëration, 3½ hr.; settling, ½ hr.; drawing, 1 hr. As soon as filling began, the air was always turned on for the purpose of blowing the precipitated sludge off of the top of the filtros plate before the full head of water, when the tank was full, made the operation more difficult. The average aëration period was 3 hr.

The third line of experiments was conducted in two glass-window inspection tanks each 1x5 ft. in plan and 10 ft. deep. The method of diffusing the air in these tanks was varied. An open jet, a filtros plate and monel metal cloth were used upon the same sewage at the same rates and at the same time. These experiments were conducted from June to the middle of November, 1915, and gave us some very valuable information.

Continuous-Flow Experiments Promising

The fourth line of experiments was carried out to test the process by the continuous-flow method. The tank used was of the same size as, and built alongside of, the tank in which the second series of experiments was conducted by the fill-and-draw process. One-sixth was divided off at one end for the sedimentation chamber, and the remainder was divided into a continuous channel about 3⅓ ft. wide and 81 ft. long, at the bottom of which were 42 filtros plates. This tank was put in operation (after securing activated sludge) the first of August and has been in continuous operation to date, with the exception of shutdowns due to the breakdown of the main sewage-pumping station.

Many series of experiments have been conducted with this tank. The volume of air has been varied from 1 to 3 cu.ft. per gal. of sewage treated; the rate from 75 to 200 cu.ft. per min.; the rate of flow from 22,000 to 74,000 gal. per 24 hr., or from a 14-hr. to a little less than a 4-hr. running-through period. Many observations have been made for controlling the sludge feed and the effects upon the process of a varying volume of activated sludge.

Another set of very interesting experiments has been conducted in an all-glass tank and in other vessels, to determine the best method of breaking up the air into small bubbles and the effect of the size of the bubbles upon the purification, also to determine the volume of air needed for oxidation and for agitation and the portion from which no results were obtained. The probable economy in mechanically agitating the liquor has also been studied.

The dehydrating of the sludge has been tried by pressing in a filter press, draining on sludge beds and by pressure due to a head of water up to 26 ft. deep. Many samples of the sludge have been analyzed and some degreased and reduced to fertilizer. Much of this work has been done by chemists and engineers connected with Swift & Co., Armour & Co. and Sulzberger & Sons, of Chicago, who have enthusiastically coöperated with us in much of our work.

The sewage treated has been taken from the main intercepting sewer, serving a population of about 275,000, to which is added the liquid wastes from the packing-house district, tanneries, breweries and other large and varied industries. Its average characteristics for the year have been 16.3 cu.yd. of solids per 1,000,000 gal. settleable in 2 hr.; 259 parts per 1,000,000 of suspended matters, varying from 150 to 600 parts; 1,109 parts of total solids; 32.9 parts of organic nitrogen; 125 parts

*Slightly condensed from a paper read before the Illinois Section of the American Water-Works Association and the Illinois State Engineers' Society, at Urbana, Ill., Jan. 27, 1916.

†Chief Engineer Sewerage Commission, Milwaukee, Wis.

oxygen consumed; 2.2 parts of dissolved oxygen and 1,161,000 bacteria per c.c. at 20° C. Sewage temperatures have ranged from 44 to 70° F. The variation of flow and strength of sewage throughout the average week is approximately 82%.

Studies of Different Types of Air Diffusers

The best method of diffusing the air has not been determined, although there are many ways available. The open air jet reduces the loss of air pressure, but reduces its efficiency, owing to the size of the air bubbles produced. This objection may be overcome by reducing the size of the orifice. We found that a $^1/_{1000}$-in. orifice would produce bubbles $^3/_2$ in. in diameter, which is below what we term the critical size. It is uncertain whether such minute orifices would not be clogged by the small particles of solid matter carried along with the air. Another objection to the open jet is its tendency to clog the air-feed pipes with sludge drawn into the pipe when the air is shut off for any reason.

The monel-metal cloth was tried for about a month only and proved quite satisfactory. Although much more porous than the filtros plate, it produced smaller bubbles, due probably to the facts that the surface of the cloth was smooth and that the air, while building the bubble, could not adhere to the cloth as it does to the rough granular surface of the filtros plate.

Kisselghur was tried out. It gave the smallest bubbles of any agent tried, but the initial frictional loss sustained in getting the air through a ½-in. plate was 20 lb., which made this diffuser impracticable.

Filtros plates, if furnished of uniform porosity, have given the most satisfactory results, but they have a few characteristics which should be realized. These plates, which are manufactured by the General Filtration Co., of Rochester, N. Y., were originally designed to filter water rather than sewage. They are 12 in. square by 1½ in. thick, composed of quartz sand, supposed to be of a uniform coefficient according to the porosity of plate required, and baked until they are quite hard and strong. The plates are produced in several different grades, which are classified in conformity with the volume of air that each plate will pass, when dry, per minute under a pressure due to 2 in. of water. These classes run from 0.5 to 12 cu.ft. of air per minute.

Table 1 shows the results obtained by using filtros, air jet, and Monel-metal cloth:

TABLE 1. COMPARATIVE RESULTS OF ACTIVATED-SLUDGE TREATMENT WITH FILTROS-PLATE, AIR-JET AND MONEL-METAL DIFFUSERS

Diffu rs	Period 1915	Pressure of Mercury, Lb.	Vol. of Air Used, Cu.Ft. per Gal.	% Bacteria Removed	Nitrate formed, P.P.M.	Stability of Effluent, Hr.
Filtros plate....	June 1 to Aug. 12	4.3	2.06	91	3.4	78
Air jet.....	June 1 to Aug. 12	3.5	1.94	91	2.2	52
Filtros plate....	Nov. 18 to Dec. 7	4.6	1.71	90	0.3	113
Monel metal....	Nov. 18 to Dec. 7	3.0	1.71	80	0.2	63

It is of the greatest importance that the agent used shall diffuse the air uniformly per cubic foot of liquor; therefore, great care should be taken in getting a diffuser of uniform porosity. So far this has not been possible with the filtros plate. Out of 780 plates tested by us for our new 2,000,000-gal. plant, where we specified a plate that would pass 2 cu.ft. of air per minute under a 2-in. water pressure and allowed a 5% variation either way, 35% of the plates received were rejected, and only 27% came within the strict terms of the specifications. The manufacturers have, since our troubles, recognized the necessity for exercising greater care and are equipped to furnish more uniform plates and to test them before shipment.

Tests made with the plates used showed that, when wet, the initial loss of pressure due to passing the air through the plate under 5 lb. pressure was ¾ lb., and for every cubic foot of air per minute per square foot of surface passing the plate there was an additional loss of ¼ lb.

Overcoming Frictional Loss in Filtros Plates

There are two ways of overcoming this great frictional loss, and the factor is an important one to consider in large plants: One is by reducing the thickness of plate to, say, ½ in. and reinforcing it with wire similar to window-glass reinforcement; and the other is by soaking the plate, before use, in a mixture of paraffin and benzine, blowing with air until the paraffin appears at the surface of the plate. This establishes through the plate fine pores, which are coated with a nonabsorbent, permitting the air to pass with the least friction; and the paraffin impels the water so that the plate does not become water-logged.

We have been operating filtros plates for nine months under all sorts of conditions and have observed no difficulty in keeping the plates from absorbing the solids in the sewage or from becoming clogged on the under surface. It may be possible for the pores to become filled in time with bacterial growths, which may be removed by forcing heated air through them.

It is necessary, however, to filter the air before delivering it to the plate, to prevent dust and dirt from reaching it. This is done by passing the air through a filter composed of excelsior and built into the feed line.

Much space has been taken up by the writer to explain the agents used for diffusing the air, as this is a very important factor in reducing the cost of the process. The efficiency of the air varies almost directly as the size of the bubbles. We call the critical diameters ⅛ in. Below this size the efficiency increases very rapidly, and the reverse is true for bubbles over this size.

There are three factors of special importance to be considered in choosing a diffuser agent: (1) The smaller the air bubble produced the greater the efficiency. (2) Greater density increases the frictional losses and, hence, the cost. (3) The tendency to absorb and retain sludge, or to permit it to reach the under side of the diffuser, tends to reduce the efficiency of the diffuser and allows sludge to precipitate on the top surface of the diffuser.

⁂

Spraying Forest Trees with insecticides is done by the Massachusetts Forestry Department at a cost as low as $6 an acre, according to a statement by F. W. Rane, Massachusetts State Forester. The department has developed spraying equipment capable of throwing a spray over tall shade trees from the ground, thus eliminating the cost of climbing the trees, so that a whole street can be sprayed in the time formerly required to treat a few trees. In the latest work, motor trucks carrying the spraying material were used instead of horses in spraying trees on highways, parks and city streets. The equipment developed in Massachusetts is also being used in the cities of Washington, Baltimore and Albany. Arsenate of lead was first developed in Massachusetts for fighting the gypsy moth and is now produced and shipped to all parts of the world. Nearly 1,000 tons of arsenate of lead was used last year as an insecticide in Massachusetts.

Machine for Placing Concrete Lining in Canals

By E. I. Davis*

SYNOPSIS—A traveling form into which concrete, mixed immediately alongside, is dumped and forced out at the bottom to form the lining of an irrigation canal. Description of the machine, its operation, its defects and advantages, as shown on an irrigation canal in Oregon.

Previous to beginning construction on the main concrete-lined canal on the West Extension of the Umatilla Project, United States Reclamation Service, Oregon, the writer asserted the feasibility of constructing a machine to place the lining. The canal generally has a capacity of 350 sec.ft., with a narrow curved invert and side slopes of 1½ to 1. The lining is 3 in. thick. It seemed like a clumsy method to dump several thousand yards of concrete into a hole and shovel it back again. Accordingly on the showing made by a small trial machine, expenditure was authorized for the construction of a large one. It was expected that if the concreting feature failed, the machine would justify itself for use as a moving templet for truing and consolidating canal banks for the reception of concrete. This work was a success from the start, although the machinery was found entirely too light to handle the concrete end.

In the spring of 1915 additional machinery was purchased and installed, and in May handwork was abandoned and the machine started. With the exception of the first few sharp curves (from 40 to 65°) the lining was machine-laid throughout for the next 11 mi.

At this point the bottom of the machine was worn out; and as a reduction in canal section occurred near-by, it was deemed wise to finish the remainder of the canal by hand, for cutting down the machine to fit the new section would have caused serious delay. During this period of operation the usual breakdowns occurred, due largely to faulty design and materials. Practically no delay resulted after the lining was continuously up with the excavation, which could not be completed fast enough. Obviously no provision was made for making repairs after regular hours.

Details of the Concreting Machine

The machine consists of a steel templet built to the exact section of the canal, which is 26 ft. wide on top and 8 ft. deep, with side slopes of 1½ on 1. The templet is about 15 ft. long on the bottom and narrower on top, with the inclined face on the front side. It is roofed over to carry a track and working platform, and the back end consists of narrow high and low hoppers and confining and surfacing plates. Within this shell thus formed the driving machinery is installed. Power is provided by an 8-hp. gas engine, driving the jig plate and dragging the machine along at a slow and uniform rate of speed. Traction is accomplished by the use of a winding drum and cable. The cable passes through a snatch block 1,000 ft. ahead, attached to a dead man in the canal bottom, and is carried back and attached to the lower right corner of the templet. This eccentric hitch permits steering by confining both lines near the machine by a forging and releasing or drawing this laterally by means of a two-ton chain block attached to an outrigger. This forging carries a sheave for the live line, the other being stationary with respect to the machine.

The machine is kept level largely by the earth compacted under it as it moves slowly along, aided by quick-acting leveling shoes on each top side. One operator and assistant keep it true to line and grade. Posts carrying two small sheaves each are used for guiding around curves.

The concrete hopper of stiffened steel plates on the rear of the machine is built entirely across the canal cross-section and about 2 ft. above the finished lining, to provide a head for easy spreading of the concrete. The

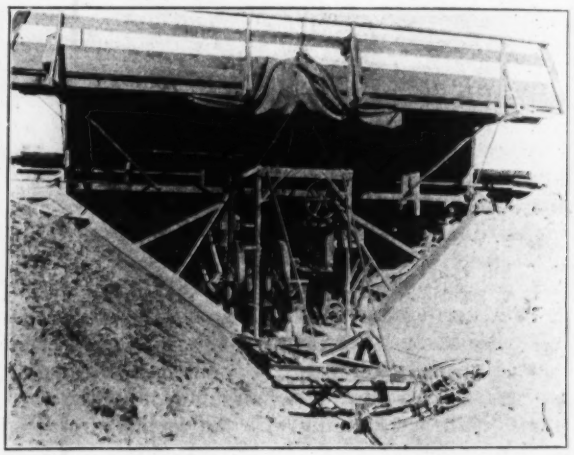


FIG. 1. VIEW OF FORWARD FACE OF MACHINE FOR LAYING CONCRETE LINING IN CANALS

canal slopes form the hopper bottom. The front face is simply a vertical bulkhead closing the rear of the machine. The back face is swung from eccentrics on a shaft carried by overhanging members. This face is tied the proper distance from the bulkhead by ⅜-in. cables spaced at frequent intervals, and this pliable connection permits the back face or jig plate to be continuously and rapidly jigged in a vertical plane. This plate extends down only to the level of the finished lining. In order to hold the concrete in the hopper on the slopes, transverse partitions of steel plate are set at certain intervals. They extend to within ½ in. of the jig plate and to nearly the level of the lining, dividing the hopper into numerous compartments and retarding the flow of the confined concrete from the higher to the lower levels. It is found entirely feasible to carry the hopper entirely full.

Immediately following the main hopper is a low one 12 in. high, also with dividing plates, to catch and hold any wet material escaping under the jig plate. This arrangement permits using a wet mix. Following this

*In charge of engineering and construction the first season and of construction the second season, Umatilla Project West Extension, United States Reclamation Service, Hermiston, Ore.

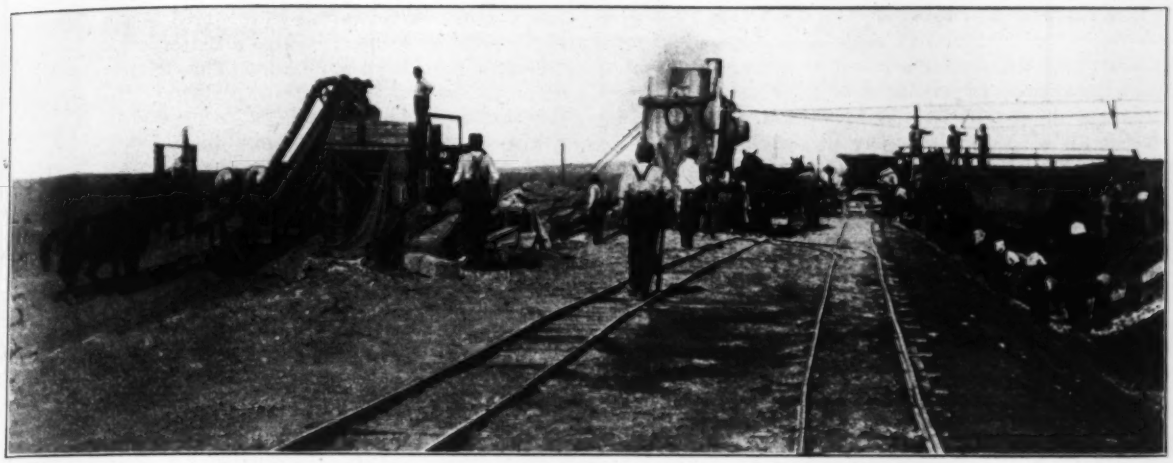


FIG. 2. VIEW SHOWING MOST OF CONCRETING PLANT IN OPERATION
From left to right—Sand and gravel screen and storage, concrete mixer, concrete wagon truck, lining machine over canal

hopper are confining and surfacing plates, hinged for easy cleaning.

The excavation for this canal was done largely on force account by scraper teams. Sand and gravel was screened into 75-cu.yd. bins located at various pits along the canal, making an average haul of over 1½ mi. From 10 to 15 six-horse teams were required for hauling, each outfit with lead and trail wagons hauling an average load of 5 cu.yd. The wagons were specially equipped with 6-in. tires. Roads were laid out to avoid heavy grades, an easy matter in the desert, and were well brushed, as the soil was sandy and dry. At stations 3,000 ft. apart along the lower bank of the canal, the proper amount of material was left on well-cleaned squares and fresnoed into piles.

A turntable, loosely attached to the machine and traveling along with it on the inner of the canal-bank tracks, is provided with inclines up which the car horses have no trouble in hauling the full loaded cars of 33-cu.ft. capacity. The cars, after turning onto the machine by hand, are dumped and run down the opposite incline. They are transferred to the return track by a low steel carriage spanning both tracks. This carriage is slid along by a car horse or by two men, as required. Switches at the mixer permit concreting by this point without interruption.

The mixer—a Smith side-loader with a capacity of 11 cu.ft. of mixed material—was raised by means of an incline and stand about 3 ft. to permit dumping into cars. By means of a specially devised steel track the charging hopper was run down the canal bank to the measuring hoppers and bin, which were built as a unit on skids for rapid moving along the line of canal.

FIG. 3. REAR OF THE CONCRETE-LAYING MACHINE IN OPERATION

The bin was filled by twin elevators driven by a 6-hp. gas engine, all mounted on skids along with a slip team runway. Six teams were used for supplying the elevators from the piles. Twenty head of stock moved this unit anywhere in good time. Along with this outfit were a blacksmith shop on wheels, an oil wagon and other accessories.

Water was supplied through a 2½-in. line by two 50-gal. duplex pumps driven by a 10-hp. gas engine mounted on a wagon, from canal water turned in at the diversion dam. Five 1-in. hose with ½-in. nozzles were played on the canal slopes ahead of the concrete machine to serve the double purpose of wetting and compacting. The time required to move the entire plant for a new setting was from 2½ to 3 hr., although during the second season one day was usually taken for this and other work.

The jig plate on the machine served several purposes. It tamped the concrete, insured a uniform feed and gave considerable additional mixing. This latter feature can be greatly increased at little expense. It also served to intermix the various carloads.

The concrete as left by the machine was usually smooth and uniform, but with occasional small defects. The entire lining was therefore easily surfaced. Joints 2½ in. deep were cut down the slopes on 8-ft. centers, the green concrete immediately covered with hop cloth and wet for five days. The total crew comprised about 250 men and the same number of horses.

The writer estimates that the saving in crew by the machine is more than 40 men for the same output by hand. Yet on the face of the returns but little saving seems to have resulted. One important reason for this, among

FIG. 4. FRONT OF THE MACHINE AND ADJACENT CONCRETE MIXER

The canal bank was double-tracked, and as fast as lining was laid the rails were moved ahead, enough being used to reach between stations. Although the canal slopes were partly water-settled, four men were used at the front of the concrete machine on each side to increase the packing of the earth by prying under the front edges with short-handled shovels as it moved along. They also finished the trimming and removed or added earth as required. This truing process required about 15 men, depending largely on the wind damage, with several two-horse fresno teams ahead. Fig. 3 illustrates very well the kind of lining placed and the general operation of the machine.

Truing the canal and concreting was of course a simultaneous process. As soon as a concrete car was run onto the machine the gate was opened and the contents dumped as it was run over and back. Three men working on a high platform behind the cars were necessary to bar loose the concrete. Four additional men handled the car. Cars could be dumped as fast as the concrete could be mixed and hauled in the five cars used. A full day's run was from 800 to over 1,000 ft. This amount of work calls for from 190 to 245 cu.yd. of concrete. The usual monthly progress was from 2 to nearly 3 mi. But for excavation difficulties, 4 mi. or more could have been laid.

others, is the fact that the machine and the rest of the concrete plant did not work full time, owing to the lack of excavated canal. This condition is largely explained by the persistent high winds and light soil, but no machine of any size can be economically operated on half time. During the first 16 days of last August, 9,600 ft. was lined at a cost well below $6 per cu.yd. A comparison of this figure with the cost the previous season indicates that the machine should have saved on labor alone 60c. per cu.yd. Soon after starting work for the past season, the engineers' estimate of saving by the machine in labor and material was $185 per day. This estimate of saving is explained by the following features: During the hand work of 1914 the loss of concrete from shoveling onto the canal banks was 14%. This result came from settlement into the canal slopes from the heavy impact of a scoop full of concrete. This loss can be largely avoided but requires a considerable period to determine safely the reduction of templet dimensions to compensate for it.

Throughout the 1914 season 2-in. gravel was used. With this size it was necessary to use plaster to the amount of 14% to secure a good finish. Careful car measurement of several days' duration at frequent intervals was made to secure these figures. The plaster used

was doubly expensive in that it is not effective in giving tangible yardage. During the 1915 season the machine handled the same size of gravel without plaster, and a larger size could have been used if well graduated. Immediately on resuming hand work late in 1915, 1-in. gravel was used, with which no plaster was necessary. The machine lays practically the required thickness without adjustment.

Defects and Advantages of the Machine

The machine as used was defective in several ways, but owing to the distance from extensive markets no attempt was made to improve it. The bottom, entirely too light in the first place, gradually worked out of shape and at one point finally wore through. This defect introduced a tremendous strain on the traction cable and drum, and frequent trouble resulted. The machinery as finally installed was effective and rugged, and in perfect condition at the end of the season. Certain improvements should produce a machine even more effective. A change in the tractive method, machine tamping, a provision for finished bank inspection, an improved method of charging the hopper, and proper construction to adapt the machine rapidly to varying-sized canals are the principal changes required.

It appears feasible to use this machine to line much larger as well as smaller canals; to use it in concrete-highway construction, inserting a plaster hopper if necessary; and to lay asphaltic concrete with the use of steam for power and heat. Its use in highway work seems particularly attractive, the additional mixing feature removing the danger from a defective batch. For this work the machine would be hardly more than a trailer for a mixer equipped with a slow traction; or better perhaps, the concrete would be supplied from mixing stations spaced perhaps a mile apart, thus doing away with excessive plant for rapid progress.

Construction of this machine was authorized by A. P. Davis, Director and Chief Engineer, United States Reclamation Service, Washington, D. C., and E. G. Hopson, Consulting Engineer for the Service, Portland, Ore. H. D. Newell is project manager at Hermiston, Ore. Much of the work in designing and building this machine was done by W. N. Rees, Hermiston, Ore., who shares with the writer the patent rights.

Two Illinois Meetings in One

A joint convention of the Illinois Society of Engineers and the Illinois Section of the American Water-Works Association was held at Urbana, Ill., Jan. 25 to 28.

Four papers dealt with the softening of water, including plants at the Chicago stockyards and for Hinsdale, Ill. This village of 3,000 people is now treating a very hard well water with lime and soda ash at a total cost of 5.9c. per 1,000 gal., with the result of almost eliminating the use of bottled water for drinking purposes and of reducing the expense for soap at laundries and for boiler compounds and boiler-repair work.

The activated-sludge process for sewage treatment was described by Dr. Edward Bartow, Urbana, Ill., and T. Chalkley Hatton, of Milwaukee, abstracted elsewhere in this issue.

An interesting example of sewage treatment for small towns is at Pana, Ill., where treatment was made necessary by stream pollution. This new plant is designed to meet the requirements of a population of 4,000. It consists of a double-compartment settling tank, automatic dosing tanks, trickling filter, secondary or humus tank and a sludge-drying bed. The cost (including land, legal fees, engineering and supervision) was about $31,000, and the cost of operation is expected to be less than $100 per year.

Oiling the black-soil roads of Illinois is keeping them in good condition at low cost for eight or nine months of the year. One of the engineers of the Illinois Highway Department pointed out that if such roads are to be maintained in good condition they must be treated annually. He advocated a light application of sand after the oil to improve the surface and suggested the possibility of applying a well-grouted brick surface on oiled roads when the traffic becomes heavy.

Concrete Pavement Unsatisfactory

Unsatisfactory experience with one-course reinforced-concrete pavement was described by Mr. Lohman, city engineer of Champaign, Ill. The poor results were due largely to insufficient care in laying and to having the work done late in the fall, when atmospheric conditions were unfavorable. The concrete was not mixed long enough and was usually too wet, because the men could handle it easier. No precautions were taken to protect the finished surface from sun and wind, except that it was sprinkled occasionally. Owing to the demands of merchants the streets were opened to traffic before the concrete was properly cured. Under such conditions the raveling and scaling of concrete was to be expected.

The difficulties of securing a permanent and economical paving on street-car tracks were outlined by R. F. Palmblade, chief engineer of the Peoria Ry. Concrete is not used under the ties, but is used under the paving, so that it incloses and retains the bed of stone or gravel ballast. Trouble due to vibration of rail joints is being eliminated by welding the rails and grinding the weld to a level surface.

Drainage and Surveying Problems

The relations of both surface and underground waters to drainage of land were presented in two separate papers. Mr. Brooks, Kansas City, discussed tile drainage as a protection of land from erosion by surface water. Mr. Chamberlain, Pittsfield, Ill., brought up the question of the escape of water from drainage ditches, etc., by seepage. He referred to investigations by the Department of Agriculture, and in this connection Prof. F. H. Newell, University of Illinois, outlined tests carried out by him while Chief Engineer of the United States Reclamation Service.

There was an unusual number of papers on surveying, mostly on the technical aspects of the subject. It was shown that surveying is becoming more and more a branch of engineering. It was suggested that the long-proposed licensing of surveyors should be embodied in a systematic scheme for the licensing of engineers.

Paul Hansen, engineer, State Water Survey, Urbana, Ill., was elected president of both societies. Edward Bartow, director of the Illinois Water Survey, Urbana, was chosen as secretary of the Illinois Section of the American Water-Works Association. The Illinois Society of Engineers reëlected as secretary E. E. R. Tratman.

A Fallacy in the Design of Retaining Walls

By Gilbert D. Fish*

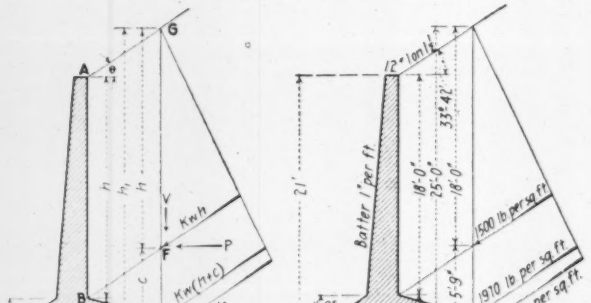
In the accepted method of analyzing the stresses in a cantilever retaining wall the vertical component of the earth thrust, as computed by Rankine's formula, is assumed to be distributed uniformly over the rear footing cantilever. The following analysis shows this assumption to be erroneous.

Let us outline the customary procedure for analyzing the rear footing of a cantilever wall to support a sloping earth bank. A tentative design of section having been made, the base pressures are calculated as follows:

The weight of concrete and its moment about the heel D (Fig. 1) are calculated; also the weight of earth in the trapezoid $ABEG$ and its moment about D. The vertical and horizontal components of the thrust of earth against the vertical section GD are obtained from the formulas[1] $V = \frac{1}{2}\, k\, w\, h_1^2 \sin\theta$, and $P = \frac{1}{2}\, k\, w\, h_1^2 \cos\theta$, the latter acting at height $\frac{1}{3}h_1$ above the base. The total vertical load on the foundation is then the sum of the weight of concrete, the weight of earth and V, the vertical component of the earth thrust. The center of pressure is at a horizontal distance from the heel, found by dividing the sum of the moment of concrete, the moment of earth and the moment of P by the total vertical load; that is,

$$\textit{Distance from heel to center of pressure} = \frac{M_c + M_e + \frac{1}{3}\, h_1 P}{W_c + W_e + V}$$

This center of foundation pressure may lie somewhat outside the middle third of the base without danger to the structure; in that case the pressure will vary uniformly



FIGS. 1 AND 2. DIAGRAMS ILLUSTRATING RETAINING-WALL THEORY

Fig. 1 (at left)—Analysis of cantilever retaining wall. Fig. 2 (at right)—Application of theory to actual wall

from the maximum at the toe T to zero at a certain distance inside the heel D. The center of pressure may, on the other hand, lie inside the middle third of TD, in which case the pressure will vary uniformly from a maximum at T to a minimum at D. In either case the variation of foundation pressure on the rear footing cantilever is readily found.

*28 West 39th St., New York City.

[1]In these formulas, k is Rankine's coefficient for active thrust of earth, and $K = \cos\theta \dfrac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}}$ where ø is the angle of internal friction of the earth and θ is the angle of inclination of the surface of the earth fill; w is the unit weight of earth.

The varying downward pressure on the cantilever, owing to the weight of earth $ABEG$ and the weight of concrete in the cantilever itself, is found in the obvious way. The effect of the vertical component of earth pressure has now to be considered. It is here that the method generally employed is at fault: the vertical component of the total earth thrust on the section GE is considered to be transferred to the footing cantilever and uniformly distributed over it.

The fallacy becomes apparent when it is observed that the earth thrust on GF is transmitted parallel to the sloping surface of the fill until it acts against the wall AB and is taken up by the concrete and steel in the vertical stem. No part or component of this thrust can be transmitted through the earth below the line FB, which is parallel to GA and to the direction of the earth thrust. The footing cantilever therefore does not carry any part of the thrust against GF. The thrust against FE is transferred to the footing along its top surface, the intensity decreasing uniformly from $k\, w\, (h + c)\, \dfrac{c}{BE}$ at E to $k\, w\, h\, \dfrac{c}{BE}$ at B; this pressure is parallel to FB, and the intensity is expressed in pounds per foot of distance along EB. To obtain the intensity of the vertical component of this pressure and at the same time to express it in pounds per foot of horizontal projection, multiply the above expressions by $\sin\theta \dfrac{BE}{b}$. We then find that the vertical pressure on the cantilever, due to the earth thrust on FE, varies in intensity from $k\, w\, (h + c)\, \dfrac{c}{b} \sin\theta$ at E to $k\, w\, h\, \dfrac{c}{b} \sin\theta$ at B, in pounds per horizontal foot. If we represent by x the horizontal distance from D to a variable point, the general expression for the vertical pressure is $k\, w\, (h + c)\, \dfrac{c}{b} \sin\theta \left[1 - \dfrac{xc}{b\,(h + c)}\right]$, or $k w \dfrac{c^2}{b^2} \sin\theta \left[\dfrac{b}{c}(h + c) - x\right]$. The shear caused by this pressure is its first integral between 0 and x, and equals $\frac{1}{2}\, k\, w \dfrac{c^2}{b^2} \sin\theta\, x \left[\dfrac{2\,b}{c}(h + c) - x\right]$. The bending moment is the next higher integral between θ and x, and equals $\frac{1}{6}\, k\, w \dfrac{c^2}{b^2} \sin\theta\, x^2 \left[\dfrac{3\,b}{c}(h + c) - x\right]$.

It is to be borne in mind that these expressions account only for the vertical component of the thrust transmitted to EB; the horizontal component affects the shears in the cantilever, but in such a way as not to matter, while it decreases the bending moments, sometimes considerably. In those rare cases where the bending appears to be the controlling element, the decrease in bending moment owing to the horizontal component may be investigated. (The foregoing remarks concerning the horizontal component may be applied as well to the horizontal reaction of the foundation along the bottom of the rear footing.)

There is one other load acting upon the footing cantilever—the earth thrust against the end ED. The horizontal component of this thrust need not be considered. The vertical component is equal to $\frac{1}{2}\, k\, w \sin\theta\, (\overline{GD}^2 - \overline{GE}^2)$ or $k\, w \sin\theta\, e \left(h_1 - \dfrac{e}{2}\right)$; this induces a constant shear

in the cantilever and a variable bending moment equal to the shear multiplied by x.

Trial Design of a Footing

This completes the analysis of the cantilever footing. The shear and bending-moment curves for any particular case can be constructed from the results. A trial design of the footing may then be tested by plotting, to the same axes and scales, the lines of allowable shear and allowable bending moment, based on the ordinary formulas for

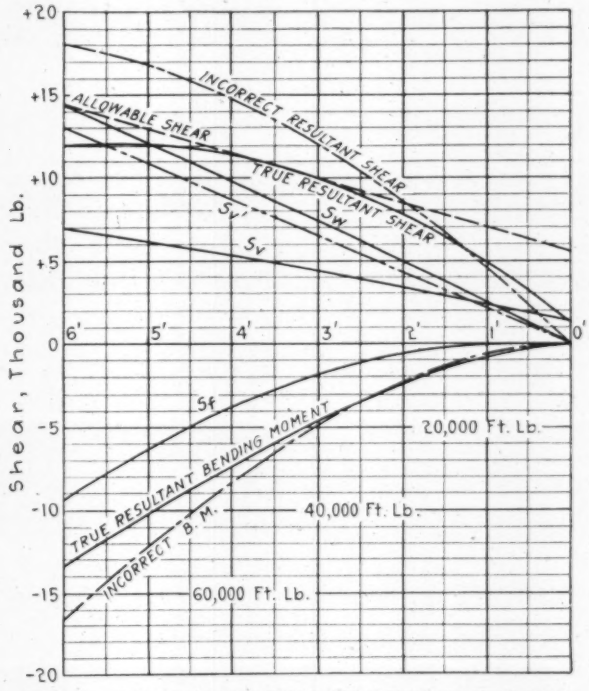


FIG. 3. DIAGRAM SHOWING STRESSES IN FOOTING CANTILEVER OF RETAINING WALL

concrete beams. If the footing cantilever has a trapezoidal section, the line of allowable shear is a sloping straight line, and the moment line is a segment of a parabola. (Notwithstanding statements to the contrary, it is insufficient to test only the section BC for bending, for the bending moment varies less rapidly than the square of x.)

The method of computation which this article is intended to discredit is not to be regarded as justifiable on the ground of simplicity, or because it errs on the side of safety, for the results are wrong, not approximate. Neither may the uncertainties of the theory of earth pressure be taken as an excuse for improper assumptions; Rankine's laws having been adopted as a basis for analysis, they must be followed consistently.

The footing cantilever of the wall shown in Fig. 2 has been analyzed for illustration. All data are given on the drawing, except the following: Unit weight of earth, $w = 100$ lb. per cu.ft.; concrete, 150 lb. per cu.ft.; angle of internal friction of earth, $\phi = \theta = 33° 42'$. It follows that $k = \cos \theta = 0.832$. The intensities of the earth thrust at several depths are shown on the figure. The total vertical load on the foundation is found to be 35,140 lb. and its center of pressure to be 7.53 ft. from D. It follows that the normal pressure between footing and foundation varies from 6,740 lb. per sq.ft. at T to zero at a point 0.60 ft. in front of D. The equation of shear for this pressure is then

$$S_f = -\frac{6{,}740}{2\,(11.0 - 0.6)}\,(x - 0.6)^2$$

The shear due to weight of footing and of earth above it is given by the expression $S_w = +\,x\,(2{,}562 - 26\,x)$. The shear due to the vertical component of earth thrust, accounting for the thrust on ED as well as that on FE, is expressed by $S_v = +21.2x\,(49.6 - x) + 1{,}408$. All of the expressions for shear, integrated between o and x, give the corresponding bending-moment equations.

In Fig. 3 are plotted curves of the shears computed from the foregoing formulas; also the curves of resultant shear and resultant bending moment. The lower of the two lines shown in dots and dashes gives the shear due to earth thrust, computed by the improper method; its equation is $S_v' = 2{,}170\,x$. The upper dot-and-dash line gives the resultant shear by the improper method; it is seen to be considerably higher than the true curve. The straight line drawn in dashes represents the allowable shear in the footing as designed, according to the expression $vbjd$, where $v = 40$ lb. per sq.in., $b = 12$ in., $j = 0.87$, and $d =$ thickness of footing minus $1\frac{1}{2}$ in. for steel protection. It will be noticed that if the footing were designed for the shears indicated by the dot-and-dash curve, it would have to be made much thicker; the resulting waste would be only partly offset by the saving from reducing the bending moments in the stem.

The two lowest curves in Fig. 3 show the resultant moments in the cantilever, the full line being according to the true method. In this case, as is usual, the bending moments are not high enough to affect the design.

Canadian Society Discusses Reorganization

The liveliest discussions at the thirtieth annual meeting of the Canadian Society of Civil Engineers (held in Montreal, Jan. 25 to 27) centered in the topics related to organization. The debates on reorganization were inspired by a series of proposed amendments to the society's by-laws; some of these were proposed by western members and some by the council of the society. All the so-called "British Columbia" amendments were lost, but the council's amendments, aimed in the same direction, were carried. The leading ones provided that (a) members must be 30 years old, engaged in engineering for 12 years and in responsible charge of work for 5 years; (b) the officers shall comprise a president, four vice-presidents, secretary, treasurer, and 24 members of the council, the secretary and treasurer to be appointed by the council and have a vote in the council; (c) the secretary shall be the general executive officer, collecting the moneys, certifying the expenditures and having charge of the society house; (d) the organization of provincial divisions is to be completed.

The outcome of all the reorganization discussion was the provision for a special committee, elected by the districts and having the apportionment of members in the same proportion as that of members of the council, to study the needs of the society and the revision of by-laws to meet the needs. The committee has to report to the council by Sept. 1; and the report goes to the membership within 30 days, so that the next annual

meeting can produce action. It is expected that this will eventually result in providing for a secretary who shall give his entire time to the society and in giving members outside the home (Montreal) district greater participation and responsibility in the society's activities.

The attendance at the annual meeting was about a third below normal—due to financial depression and members being in military service. The entertainment features were confined to a smoker, an informal dinner and one excursion.

The report of the treasurer and finance committee showed the evil effects of hard times and war on the treasury. The proposed 1916 expenditures have been cut from $20,000 to $16,600, to meet an expected drop in income from $22,000 to $15,000. The report of the council showed a total membership of 3,076, of whom 363 had entered military service and 13 had been killed.

Satisfactory progress in timber- and water-resources investigations and in the acceptance of town-planning, also the discovery of phosphate, were reported by the Conservation Committee.

A specification for portland cement was reported by a special committee and adopted. The features are: A fineness rejection requirement of 8% on a 100 sieve and 25% on a 200 sieve; initial set after 30 min. and final set after 1 hr. and within 10 hr.; tensile strength of neat cement, 175 lb. per sq.in. after 24 hr., 500 lb. in 7 days and 600 lb. in 28 days. It will be noticed that these requirements are the same as those adopted by the American Society for Testing Materials, the standard American specification. The Canadian specification follows the American in intent though not in wording.

The Committee on Steel-Bridge Specifications reported progress in framing standard specifications and hoped to present a final report in another year. The Committee on General Clauses in Specifications, which had had its 1915 report referred back to it, reported that it could not formulate clauses generally applicable to all sorts of work and could only prepare them for a few specific fields like excavating, structural, etc. It was asked to proceed on such lines.

The Committee on Roads and Pavements presented a scheme for classifying traffic on roads—light and heavy horse-drawn steel-tired vehicles; rubber-tired pleasure automobiles and motor trucks; steel-tired self-propelled vehicles; light, medium and heavy travel. Forms were recommended for reporting the history of pavements and the traffic thereon.

The Committee on Sewage Disposal and Sanitation reported a plan for improving provincial public-health acts by requiring three engineer members on the provincial boards, by requiring that plans for sewerage and waterworks be approved by provincial boards, by outlining the necessary scope of the plans submitted and by giving the boards jurisdiction over operation.

The society adopted a resolution urging on the Dominion Government coöperation with the society in the proper development of the engineering branches of military service, asking that engineers be not drafted into other branches of service and that adequate engineering training and experience be required.

The society elected as its president for the year 1916 G. H. Duggan, Chief Engineer of the St. Lawrence Bridge Co., Montreal; and as vice-president, T. H. White, Chief Engineer, Canadian Northern Railway.

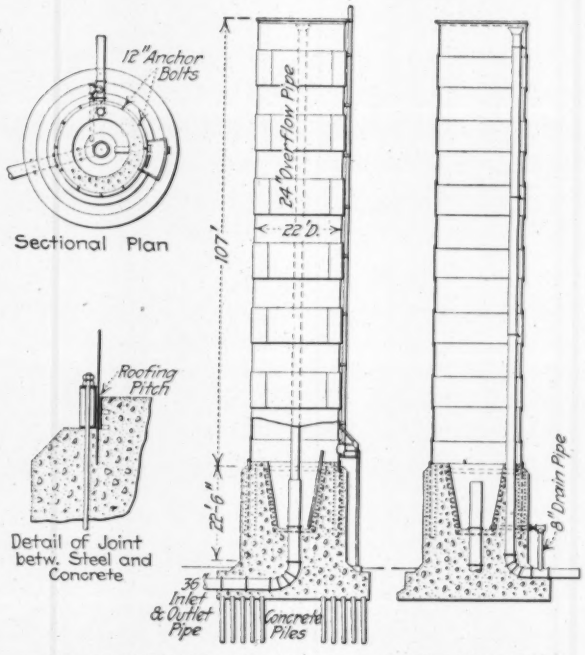
A Composite Standpipe

A steel standpipe on a reinforced-concrete base has been erected at the Steelton, Penn., plant of the Pennsylvania Steel Co. The height to which the concrete base was carried was fixed by the limit to which material could be raised economically with the equipment at hand. The base is reinforced to prevent bursting and to obviate shrinkage cracks.

Wind pressure is resisted by means of twelve 1¾-in. anchor bolts passing through angle lugs riveted to the shell and extending down into the concrete, as shown in the sketch. The concrete in the portion of the base subjected to the direct pressure of the water was made of four parts of crushed limestone, two parts of limestone screenings and one part of cement.

To prevent cracking from too rapid drying, the hollow was filled with water and the outer surface was kept wet for several days after the forms were removed. To guard against leakage from a possible separation of the shell from the concrete on account of difference in expansion of the two materials, hot asphaltum was placed between the shell and the concrete.

C. P. Turner, Chief Engineer of the Pennsylvania Steel Co., stated in the "Journal" of the Engineers' Society



STEEL STANDPIPE ANCHORED TO CONCRETE BASE

of Pennsylvania that with the standpipe filled there was scarcely an indication of seepage through the concrete and none whatever between it and the shell or the pipes. Beyond care in mixing and in placing the concrete, no waterproofing was attempted.

⁂

Hollow Concrete Blocks have been largely used in the construction of Government buildings for the Panama Canal. A total of 230,000 blocks had been made in the concrete-block plants at the end of 1914. Most of the blocks turned out are 12x12 in. by 3, 4, 6 and 8 in., but a few 12x16x8-in. blocks have been made. The average cost per block of about 47,000 blocks turned out by the plant in April, May and June, 1915, was 8.7c. per block. This cost includes an arbitrary charge of 3c. for interest, depreciation, etc., on the plant used in manufacture.

Bituminous-Carpeted Concrete Road of New Type

There have been many attempts made to cover concrete pavement with a thin carpet coat of bituminous material, but the results have been very unsatisfactory, with the single notable exception of the asphalt-coated concrete roads built by the California Highway Commission. The most recent and apparently one of the most successful attempts to build a bituminous-carpeted concrete pavement is the work done by the Hassam Paving Co., of Worcester, Mass., which uses a patented process under the trade name of "Bi-co-mac." Russell St., in the city of Worcester, was paved in this manner in 1914 and is in very satisfactory condition. It is a residence street bordering Elm Park, and the traffic is not heavy.

Rolling Hassam-grouted base | Applying the Bi-co-mac surface | View of finished pavement

LAYING BI-CO-MAC CONCRETE PAVING ON A STREET IN PORTLAND, MAINE

The new pavement construction is covered by eight patents, six of which were issued in 1906 and 1907 to Walter E. Hassam, of Worcester, and two in 1912 to William T. Headley, of Philadelphia. The former deal with the construction of the concrete base and the latter with the bituminous top.

The concrete base is of cement-grouted broken stone made by placing a layer of stone on the rolled subgrade, thoroughly rolling it and then filling the voids with portland-cement grout and rerolling the wet mass to consolidate it. This process is covered by the Hassam patents 819,652 and 851,625. The subsequent patents—861,650; 861,651; 890,902 and 912,125—cover the rolling in of a top course of screenings, a reinforced pavement of the same type and the construction of a grout-mixing machine.

The top wearing surface, about an inch thick, is a mixture of cement concrete and an emulsified asphaltic cement. The concrete is mixed with an excess of stone or other coarse aggregate so that the mortar does not fill all the voids. The remaining voids are filled with the emulsified asphalt. The resulting material is claimed to be a resilient waterproof concrete, which it is apparent is a hybrid between ordinary cement concrete and asphaltic concrete.

The mixture for the top coat consists of the following: 1 cu.yd. of ¾-in. crushed rock, ½ cu.yd of clean sand, 166 lb. of portland cement and 57 gal. of asphalt emulsion known as Bi-co-mac, diluted with 20% of water; the percentages are 61, 27, 4 and 8 respectively. These may be mixed in the same mixer as used for the cement grout, which is a horizontal trough with revolving paddles on a horizontal shaft. The cement, sand and stone are mixed dry and the asphalt emulsion added, or according to the patent specifications, the stone may first be covered with the asphaltic cement and the sand and portland cement subsequently added. The wet mixture has about the consistency of ordinary cement concrete.

The asphaltic cement consists of asphalt heated and fluxed with the necessary amount of asphaltic oil. About 80% of this composition is mixed with 20% of cotton-seed oil, caustic potash and water to form an emulsified asphaltic cement, which can be diluted to any extent with cold water and mixed cold with the concrete aggregates. This combination is covered by patents 1,035,787 and 1,045,286, issued to William T. Headley, of the Headley Good Roads Co., Philadelphia, in November, 1912.

The Hassam concrete base is constructed as described above, except that instead of flushing the stone with grout the grout is brought to within ¼ in. of the top of the stone. On top of this the 1-in. wearing surface is placed before the concrete base has set, and then the pavement is rolled with a steam roller. If this final rolling fails to flush sufficient asphaltic cement to the top to produce a tight surface, a flush coat of the Bi-co-mac is applied and covered with screenings or coarse sand. Traffic must be kept off the finished pavement for at least five days.

A cross-section of the completed pavement shows an irregular line between the cement concrete base and the black asphaltic-concrete surface coat. The pavement is claimed to combine the good qualities of both cement concrete and asphaltic concrete. The cost is apparently not much in excess of plain concrete. The pavement in Russell St., Worcester, cost $1 per sq.yd., but this was a special price in order to introduce the pavement.

Anthracite-Coal Shortage—The United States Geological Survey publishes the opinion that a 2½% reduction in the Pennsylvania anthracite output (79,100,000 tons in 1915 against 81,100,000 tons in 1914) is due to the fact that "consumers have not laid in their winter supply in advance to the same extent as formerly, and the retailers have been equally laggard in placing their orders," this in spite of warnings of a coming scarcity. The discount rates for summer purchase did not stimulate the same amount of advance buying in 1915 as in previous years. The capacity of the mine storage yards has been taxed to the utmost to take care of the surplus output.

Flood Control in Los Angeles County in California--I

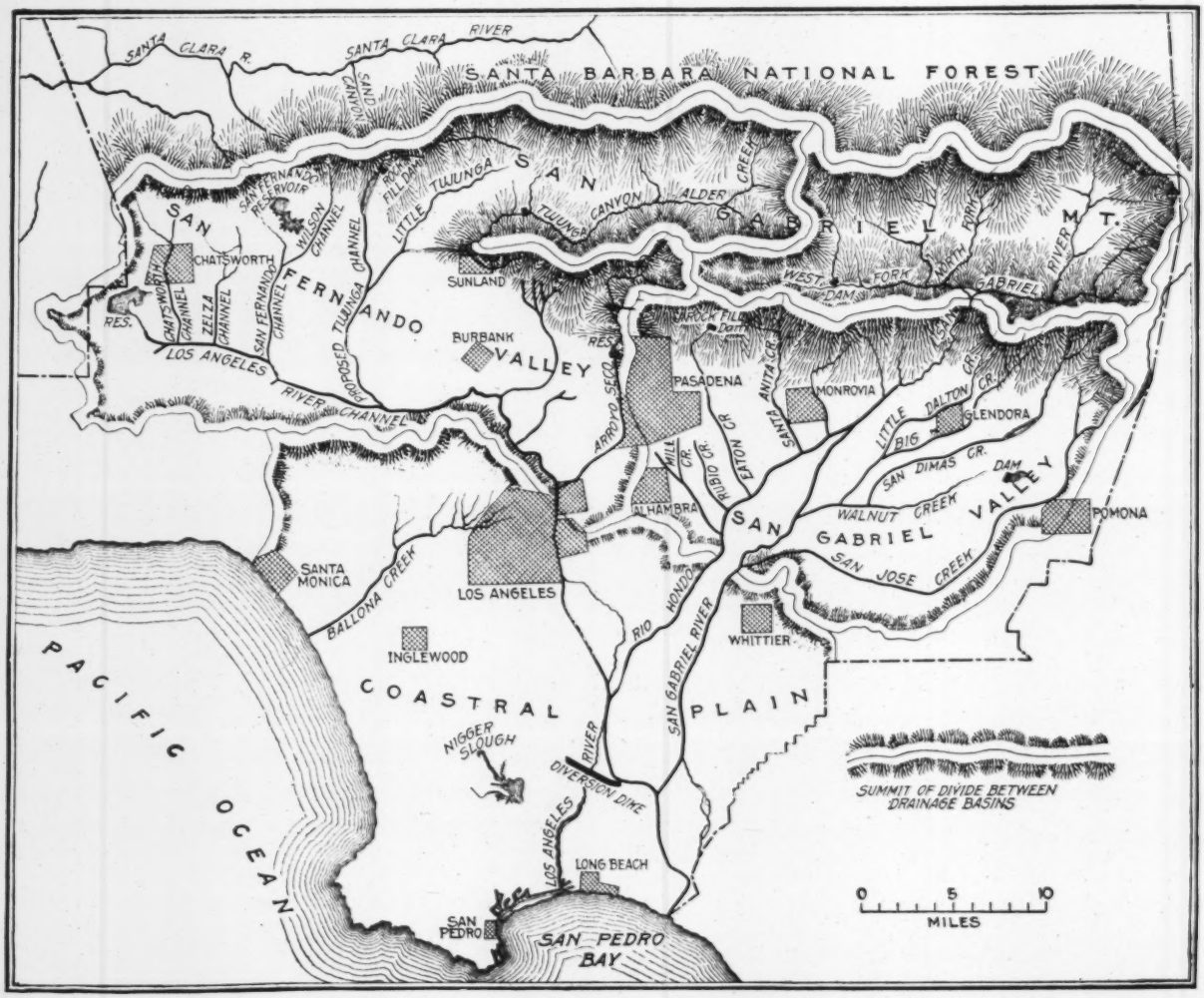
SYNOPSIS—The steep mountains that border the plains on which stands the City of Los Angeles are subject to heavy rainfalls during the winter months. The result is a very heavy flood discharge, which in the winter of 1914 did over $10,000,000 worth of damage in Los Angeles and the country around it. Another serious flood occurred on Jan. 18, 1916. A board of engineers has recommended works for flood prevention on different lines from any hitherto adopted in the United States, including check dams in the mountain valleys and works for spreading the water on the débris cones at the mouth of the mountain cañons.

In February, 1914, a disastrous flood visited the region around Los Angeles, Calif., causing damage estimated at over $10,000,000. In April following, the Board of Supervisors of the county appointed a board of engineers to investigate the causes producing the flood and report upon what scheme should be adopted to protect against similar casualties in the future. The engineers appointed on this board were H. Hawgood, Charles T. Leeds, J. B. Lippincott and F. H. Olmsted. Since this appointment the board has been engaged in extensive studies, surveys and experimental work, as a result of which it has recommended for adoption a plan of flood protection for Los Angeles County involving an estimated expenditure in round numbers of $16,500,000.

The conditions producing floods in southern California are so unusual and the methods proposed for overcoming them are so unprecedented, in American engineering practice at least, that a full description will be of interest.

Floods Frequent in the Past

To those who are unfamiliar with the climatic conditions of southern California and who have only the limited knowledge that the region is in the arid zone, where irrigation is necessary to agriculture, it may seem strange



SKETCH MAP OF PART OF LOS ANGELES COUNTY, CALIF., SHOWING MOUNTAINS AND PLAINS AND GENERAL LOCATION OF CHANNELS AND WORKS RECOMMENDED BY BOARD OF ENGINEERS ON FLOOD CONTROL

that this section should be a serious sufferer from floods. The records show, however, that the disastrous flood of 1914 was not an isolated phenomenon by any means. A study of the records of past high water shows that floods in Los Angeles County, causing more or less damage, have occurred at average intervals of 3¼ years in the past half century. In some of the flood years more than one high-water period has occurred. In 1889-90, for instance, there were four severe floods, one of which approached the record in height.

The volume of water discharged by these rivers in flood, moreover, surprises one who knows that the river beds are wholly dry for a large part of the year and that the average total annual rainfall at Los Angeles for a period of 30 years averages less than 16 in. One of the streams to be controlled, the San Gabriel River, had a flood discharge estimated at 27,000 cu.ft. per sec. in 1914 and 47,000 cu.ft. per sec. in the great flood of 1884—and this from a drainage area only 220 sq.mi. in extent.

High Mountains Shed Water on the Plains

The topographical and climatic conditions that give rise to this remarkable flood situation are very interesting and unusual. The City of Los Angeles and its suburbs lie on a gently undulating plain that slopes back from the ocean a distance of some 25 mi., with a gradual rise of 3 or 4 ft. per mi. near the ocean and 15 to 20 ft. per mi. 20 mi. away, at Los Angeles. From the edges of this plain rise a group of steep mountains, their summits reaching an elevation of 5,000 to 8,000 ft. seamed and cut by innumerable valleys. Some 750 sq. mi. of this high mountain area drains onto the plains below.

The torrents which flow down these mountain valleys when heavy rains occur have a velocity produced by slopes averaging 200 to 300 ft. per mi. on the larger streams and 700 ft. to 1,400 ft. per mi. on the smaller streams. The sides of the mountains are mostly covered with a dense growth of low shrubs and chaparral, but large areas have been denuded by forest fires. The soil on these slopes contains much fine material as well as sand, gravel and rock, and at every flood a great volume of suspended matter is carried down by the streams.

Relation of Rainfall to Floods

The average annual rainfall recorded by the United States Weather Bureau at Los Angeles for the 38-year period from 1877 to 1915 is only 15.81 in. On the average most of this annual rainfall comes in the four months from December to March. The average rainfall for January is 3.16 in. and the average for February, 3.17 in. This rainfall, however, is subject to great fluctuations in different years. In the year 1883-84, for example, the rainfall was 38.18 in., and in the year 1889-90 the rainfall was 34.84 in. In both these years high floods were recorded. Such high annual precipitation, however, is not necessary to produce floods, for in the great flood year of 1914 the total rainfall in the season was only 23 in., but 10.35 in. of this fell in January and 7.04 in. in February.

It should be clearly understood, however, that the record of the rain gage in Los Angeles at an elevation of some 200 ft. above sea level is little indication of the rainfall upon the mountains 10 to 20 mi. away at elevations of 1,000 to 8,000 ft. The winter rainfall on the Pacific Coast is caused by the moist winds from the Pacific as they sweep over the land. Oftentimes these winds will sweep across the low plains near the ocean without precipitating a drop of moisture, but will let loose torrents of water as they strike the cold mountainside at high elevation. Rainfall records in San Diego County, for example, 150 mi. farther south, show that with an average annual rainfall of less than 10 in. at the coast, pre-

CROSS-SECTION NO. 1

Condition of Channel—Both banks are composed of granite boulders from 4 in. to 2 ft. in diameter. The bottom of the channel is sand and gravel with rocks up to 3 in. in diameter. The surface of the water contains small eddies. Computed value of n from field data is 0.02815; approximate value, 0.030

CROSS-SECTION NO. 2

Condition of Channel—The west bank is composed of boulders from 2 in. to 1 ft. in diameter, the average diameter being about 4 in. The boulders on the east bank range from 4 in. to 3 ft. in diameter, the average being about 1 ft. The bed of the channel is covered with boulders from 4 in. to 2 ft. in diameter. The surface of the water is full of small waves. Computed value of n is 0.0354; approximate value, 0.035

CROSS-SECTION NO. 3

Condition of Channel—Both banks are very irregular and are composed of granite boulders from 4 in. to 2 ft. in diameter. Numerous large rocks appear above the surface of the water. The bottom of the channel is covered with boulders from 6 in. to 2 ft. in diameter. The surface of the water is full of eddies and waves. Computed value of n is 0.0416; approximate value is 0.04

CHANNEL OF SAN GABRIEL RIVER MEASURED FOR COMPUTING COEFFICIENT OF ROUGHNESS

CROSS-SECTION NO. 1—LOS ANGELES RIVER, APR. 28, 1915

Taken about 200 ft. below filtration galleries; channel sandy; scattered piles of débris up and down the stream; banks firm on east side, but showed signs of having receded on west side. Information obtained from an employee of the Los Angeles Water Department shows that the bed of the channel at this point is much lower now than before the flood. Width of channel, 267 ft.; wetted perimeter, 275 ft.; slope, 0.0035; "C," 59.0; discharge, 24,420 sec.-ft.; area of cross-section, 2,380 sq.ft.; hydraulic radius, 8.65; "n," 0.0375; velocity, 10.27; drainage area, 334.8 sq.mi., of which 242 are mountainous and 92.8 are valley; run-off per square mile, 73.2 sec.-ft.

cipitations as high as 50 and 60 in. per annum are recorded on the mountains at an elevation of 6,000 to 7,000 ft., only 40 or 50 mi. away.

In the great storm of February 18 to 21, 1914, in Los Angeles County the rainfall at San Pedro, on the coast, was only 2.03 in. At Los Angeles, 20 mi. inland, it was 7.07 in. At Mount Lowe, 11 mi. from Los Angeles, at an elevation of 5,000 ft. it was 19.20 in. Where the rainfall is subject to such variation within short distances it is better to obtain data as to the flood discharge of streams by other methods than computation of precipitation and runoff.

Manifestly, it was not possible to measure the floods while they were actually in progress. However desirable it might have been to do this, it would have been practically impossible, even had the need for this data been realized at the time and an organization been ready to secure it. The flood so interrupted all means of transportation over the great area which it covered that it would have been practically impossible for engineers to travel over the territory and reach observation points while the flood was at its height. A still further and sufficient reason is that the violence of the torrents was so great that current-meter measurements would hardly have been possible, even if engineers could have reached the different streams.

The board employed J. L. Norton to examine each one of the principal streams, measure the cross-section of the channel at various points as determined by the high-water mark and also determine the hydraulic slope, or gradient, of the flood. It is understood, of course, that this method of computing flood discharges is at best but an approximation. A stream when in violent flood will considerably change its cross-section. While the flood is at its height, a large amount of débris rolls down its channel. As the flood begins to subside, this débris will be deposited in the bed; and usually at this period the stream will most violently cut its bank. Consequently, the cross-section left after the flood has passed may be quite different from that which existed at the time of maximum flood discharge.

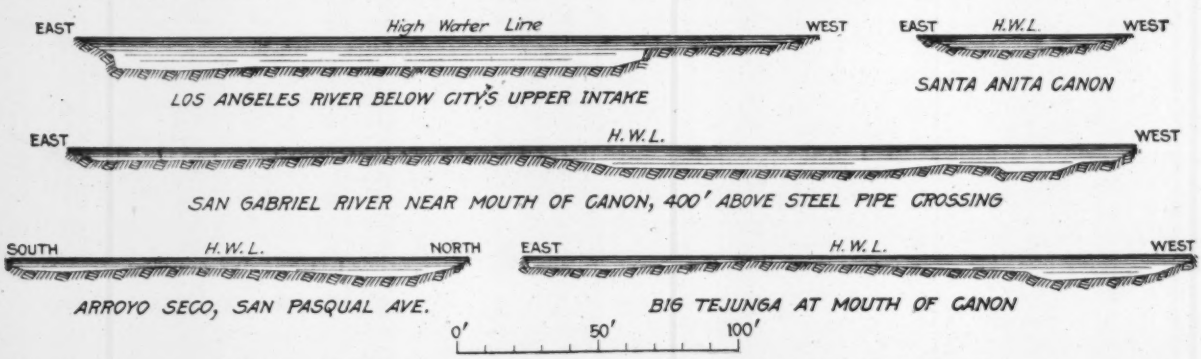
In order to determine with some degree of accuracy the value of the coefficient n for rough boulder-covered channels, to be used in computing flood flows by Kutter's formula, three typical sections with as uniform channels as possible were selected on the San Gabriel River near

CROSS-SECTION NO. 7—ARROYO SECO, MAY 4, 1914

Taken at north end of San Pasqual Ave.; bed of channel rough, being covered with coarse gravel and boulders; maximum velocity within 80 ft. of the north bank; both banks cut back by the current. Width of channel, 163 ft.; wetted perimeter, 169 ft.; slope, 0.009; "C," 45.0; discharge, 7,610 sec.-ft.; area of cross-section, 810 sq.ft.; hydraulic radius, 4.84; "n," 0.045; velocity, 9.4; drainage area, 42.7, of which 30 are mountainous and 12.7 are valley; run-off per square mile, 178 sec.-ft.

Azusa, Calif. The selected channels are shown in the accompanying photographs. The line drawn across the foreground indicates the middle of the longitudinal section over which the slope of the water surface was measured.

The cross-section of the channel was measured with a Y-level, rod and tape. The slope of the water surface was determined by leveling between sharp-cornered rocks placed in stilling boxes located equal distances above and below the cross-section used for computing the hydraulic elements of the stretch of channel in question. The velocity of the stream flow was measured with a current



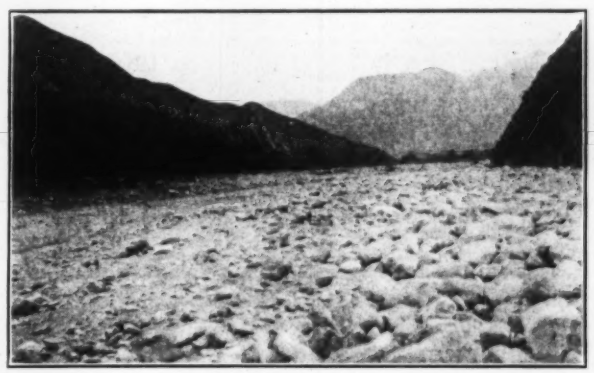
FLOOD CROSS-SECTIONS OF VARIOUS STREAMS IN LOS ANGELES COUNTY

meter by wading. The channels in all cases were free from brush.

The value of n thus determined was used in computing the flood flows in other streams with similar rough bottom, as well as in the San Gabriel. The entire engineering board conferred in deciding on the value of n to be adopted for each stream, being aided by photographs of the section and the advice of the engineer who had measured the cross-section.

In most cases the slope of the stream was determined by observing high-water marks on the banks at distances 800 to 1,000 ft. apart and then determining the fall and the distance between these marks by direct measurement. In addition, as a check, the actual slope of the bed of the stream was measured by leveling.

The results of the computations made according to the described method were carefully studied and compared, and where observations gave discordant results they were eliminated. In this way it is believed that the final figures printed in the accompanying table give a fair approximation of the actual flood discharges.

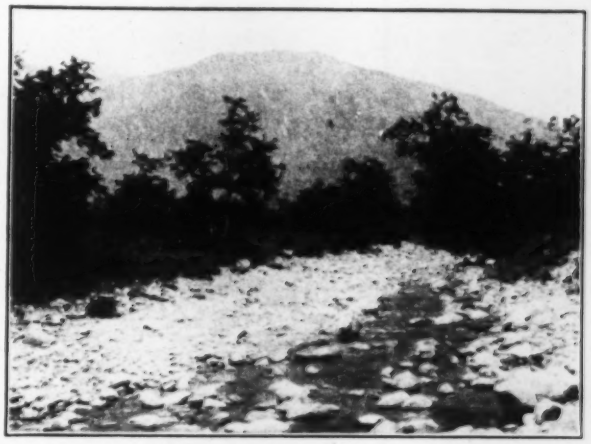


CROSS-SECTION NO. 9—BIG TEJUNGA, MAY 5, 1914

Taken at the mouth of Big Tejunga Cañon; channel rough and strewn with boulders; south bank is of rock and precipitous. The main current followed the north bank, which has been cut back. The maximum velocity was within 100 ft. of the north bank. Width of channel, 240 ft.; wetted perimeter, 246.5 ft.; slope, 0.0124; "C," 52.5; discharge, 13,600 sec.-ft.; area of cross-section, 1,068 sq.ft.; hydraulic radius, 4.74; "n," 0.0375; velocity, 12.73; drainage area, 118 sq.mi., all of which is mountainous; run-off per square mile, 115 sec.-ft.

ESTIMATED FLOOD DISCHARGES FROM STORMS OF JANUARY AND FEBRUARY, 1914

Stream	Drainage Area, Sq.Mi.	Value of n	Discharge in Cu.Ft. per Sec. Total	Discharge in Cu.Ft. per Sec. Per Sq.Mi. of Drainage Area	Character of Basin
Los Angeles River Watershed					
Big Tejunga	118.0	0.0375	13,600	115	All mountainous
Sunland Wash	6.5	0.0275	4,620	712	2.7 sq.mi. valley; 3.9 sq.mi. mountainous
Little Tejunga	22.3	0.03	4,100	184	All mountainous
Paccima Creek	30.0	0.05	5,400	180	All mountainous
Verdugo Creek	21.9	0.03	7,700	352	
Arroyo Seco at Devil's Gate	30.5	0.025	11,400	374	27 sq.mi. mountainous; 3.5 sq.mi. valley
Arroyo Seco at San Pasqual Ave.	42.7	0.045	7,610	178	30 sq.mi. mountainous; 12.7 sq.mi. valley
Los Angeles River below Lankershim bridge	304.3	0.0375	16,780	55.2	220.8 sq.mi. mountainous; 83.5 sq.mi. valley
Los Angeles River below Main St. bridge	534.1	0.025	31,140	58.2	363.1 sq.mi. mountainous; 171 sq.mi. valley
San Gabriel River Watershed					
Eaton Wash	12.5	0.035	4,590	367	10.3 sq.mi. mountainous; 2.2 sq.mi. valley
Santa Anita	18.17	0.045	3,168	174	All mountainous
Sawpit Canyon	7.37	0.05	4,070	550	All mountainous
Big Dalton	7.56	0.05	1,070	142	All mountainous
Little Dalton	3.34	0.05	1,018	305	All mountainous
San Gabriel River	228.68	0.035	26,680	117	All mountainous



CROSS-SECTION NO. 14—SANTA ANITA WASH, MAY 11, 1914

Taken about 500 ft. above the Foothill Boulevard, below where the Little Santa Anita joins the Santa Anita. The banks are low and lined with brush and trees; the bed is rough, containing large boulders. Width of channel, 78.5 ft.; wetted perimeter, 79.5 ft.; slope, 0.0142; "C," 42.0; discharge, 3,168 sec.-ft.; area of cross-section, 312.2 sq.ft.; hydraulic radius, 4.002; "n," 0.045; velocity, 10.02; drainage area, 18.17 sq.mi., all of which is mountainous; run-off per square mile, 174 sec.-ft.

The photographs herewith are taken from the report, showing typical cross-sections of flood channels. The accompanying table also gives the final figures of runoff during the 1914 flood for the various streams measured in cubic feet per second per square mile of drainage area.

It will be seen that some of the smallest streams showed a runoff of 300 to over 700 cu.ft. per sec. per sq.mi., while the larger streams (except the Los Angeles River, from which, because of the large area of plain included in its watershed, the flood wave is much longer) showed about 115 cu.ft. per sec. per sq.mi.

Comparison of the figures in the table with those estimated for the flood of last month (given on page 239 of *Engineering News* of Feb. 3) shows a flow nearly as great in the Arroyo Seco as in 1914, but only about half as great a flow in the Los Angeles River, which has a longer flood wave.

[*The second part of the abstract of the report will be published in an early issue.*]

CROSS-SECTION NO. 18—SAN GABRIEL RIVER, MAY 12, 1914

Taken below the mouth of Rogers Cañon. The bed of the channel is rough with gravel and boulders 1 ft. to 18 in. in diameter. The banks are of light soil and have been cut back, but I could not tell whether the cutting was done in February or during other floods. Width of channel, 383 ft.; wetted perimeter, 386 ft.; slope, 0.0079; "C," 58.0; discharge, 26,680 sec.-ft.; area of cross-section, 2,178.2 sq.ft.; hydraulic radius, 5.64; "n," 0.035; velocity, 12.25; drainage area, 228.68 sq.mi., all of which is mountainous; run-off per square mile, 117 sec.-ft.

Field and Office

Trolley-Car Concrete Mixer for Street-Railway Track

The Birmingham Railway, Light and Power Co., Birmingham, Ala., has in service two concrete mixers mounted on trolley-car trucks. They are used for practically all the inter-track paving of the company's urban car lines. One of these mixers in service is shown in the accompanying views.

A great deal of concreted track, of the type shown in Fig. 1, is being placed in Birmingham. This section com-

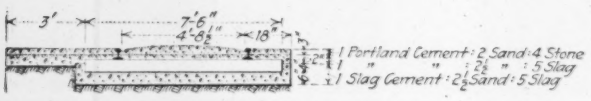


FIG. 1. TYPICAL HALF-SECTION OF CONCRETED STREET RAILWAY TRACK, BIRMINGHAM RAILWAY, LIGHT AND POWER CO.

prises a 10-in. bottom layer of 1 : 2½ : 5 concrete, in which local slag cement and a broken slag aggregate are used; a 4-in. layer of 1 : 2½ : 5 portland-cement and slag aggregate concrete, in which the top of the timber tie and the bottom of the rail are embedded; and a 3-in. finish coat of 1 : 2 : 4 portland-cement and stone concrete, crowned to the street surface.

In laying this pavement the material is brought into the street in wagons or on trolley cars and spaced in piles at intervals along the line. The mixer-car, traveling on one of the tracks to be paved, then takes the material from the piles and transforms it into the concrete to be dumped into the immediately adjacent tracks.

Each mixer-car consists of an ordinary two-truck car frame mounting a 1-yd. Smith mixer, with loading and delivery equipment. There is a regular street-car motor for both driving the mixer and propelling the car. The loading trough, Fig. 2, can be let down to the street surface and filled directly from barrows. Fig. 3 shows the mixer working along a track to be concreted and in the foreground the finish coat is being screeded.

L. L. Newman is Superintendent of Way and Structures of the railway.

Trap vs. Slag for Aggregate

Comparative tests of trap rock and slag as concrete aggregates were made during the last year for the National Slag Co., of Newark, N. J., at the department of civil engineering of Columbia University under the direction of Frederick Miller. The results for 1, 3, 6 and 12 mo. are given in the accompanying tables.

TABLE 1. ULTIMATE COMPRESSION STRENGTH OF 8x16-IN. CONCRETE CYLINDERS, IN LB. PER SQ.IN*

	28 Days	3 Mo.	6 Mo.	12 Mo.
Slag concrete	2,465	3,496	3,567	4,187
Trap rock concrete	1,975	2,961	3,053	3,537
Higher compressive strength of slag concrete	490	535	514	650
Percentage higher compressive strength of slag concrete	24.8	18.0	16.9	18.4

*The averages given are the results of testing four specimens of both types of concrete at each period.

TABLE 2. MODULUS OF ELASTICITY OF SLAG AND TRAP ROCK AND TRAP ROCK CONCRETE THREE MONTHS OLD*

	Lb. per Sq.In.
Slag	4,126,000
Trap rock	4,036,000

*The specimens tested were three slag and three trap rock concrete 8x16-in. cylinders. An average value in each case was taken up to a unit load of 650 lb. per sq.in.

The average weight of slag concrete tested was 140 lb. per cu.ft., while the average weight of trap concrete was 151 lb. per cu.ft. The slag weighed 80 lb. per cu.ft., and the trap weighed 100 lb. per cu.ft.

The traprock was from the Hudson Palisades. It was separated into ¾, ½, ¼-in. sizes and "dust" and was artificially combined so that the grading was identical with that of the slag, which was used as received from the National Slag Co. The sand used was Cow Bay, a

FIGS. 2 AND 3. CONCRETE MIXER MOUNTED ON TROLLEY CAR TRUCK CONCRETING INTER-TRACK SPACE OF STREET RAILWAYS AT BIRMINGHAM, ALA.

typical Long Island sand, and the cement was a mixture of three brands of portland cement, as approved by the New York Building Bureaus.

The mixture of the concrete was in all cases 1:2:4 by volume, and it was mixed in a batch mixer. In making up the complement of stone for each batch, proportional amounts of each aggregate were carefully measured by weight. It was thought that this procedure would insure greater uniformity in results.

"Dapple" Finish for Concrete

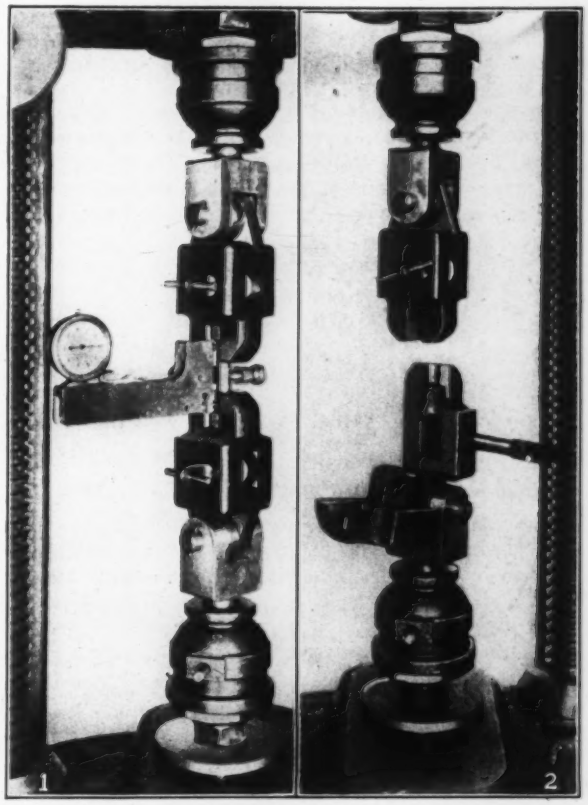
In the construction of dry dock No. 1 for the Panama Canal all the horizontal surfaces of the dock, including the coping, altars and floor, were given what is called a "dapple" finish. The method of producing this finish is described as follows by the Engineer of Terminal Construction, H. H. Rousseau, in his annual report just issued:

The vertical surfaces of the dock were given no special treatment during placing of the concrete other than to work the mortar to the face of the form. All horizontal surfaces in the dock, including coping, altars and floor, were covered to an average depth of from 1½ to 2 in. with mortar composed of cement and specially sifted sand mixed 1:2. This mortar was applied at the same time the concrete was placed.

These horizontal surfaces were all given a dapple finish in the following manner: After applying the mortar and thoroughly rodding it until the water had been worked to the surface and swept off the block with a rodding stick, the surface was thoroughly floated with a wooden float. Following this, a wooden float was used to produce the rough or dapple surface by placing the float squarely on the surface, pressing it slightly and lifting it directly upward from the surface. In this way the cement surface adhering to the float is slightly raised so as to give the appearance of chipped stone. This dapple finish is sufficiently rough to prevent slipping and makes a very suitable and satisfactory working surface.

Grips for Plain-Shouldered Tension Specimens

The usual standard 2-in. test piece for forgings and castings has threaded ends that screw into the grips. This test piece requires considerably more time to make and therefore costs more than one with plain enlarged ends. Specially designed grips used in the Pennsylvania R.R. laboratory at Altoona, Penn., take such plain specimens. They require only the turning of the reduced section to proper diameter and filleting of the shoulders. Usually the ends have to be turned to get them into the space provided in the grips.



FIGS. 1 AND 2. PENNSYLVANIA R.R. GRIPS FOR 2-IN. PLAIN SHOULDERED SPECIMEN

At left, closed, with Berry gage on specimen; at right, grips opened; ball-and-socket adjustment above and below grips

One very marked advantage of this Pennsylvania R.R. grip over others in use is that it permits the use of an extensometer or dividers on the 2-in. gage length while pulling. (One of the photographs shows a 2-in. Berry gage in place.) In other types only that part of the specimen between the jaws can be got at. A modification used by the Union Steel Casting Co. has more space inside the jaws to accommodate the enlarged section of the end of the test piece. The space will take care of a shouldered section 4 in. long. As a

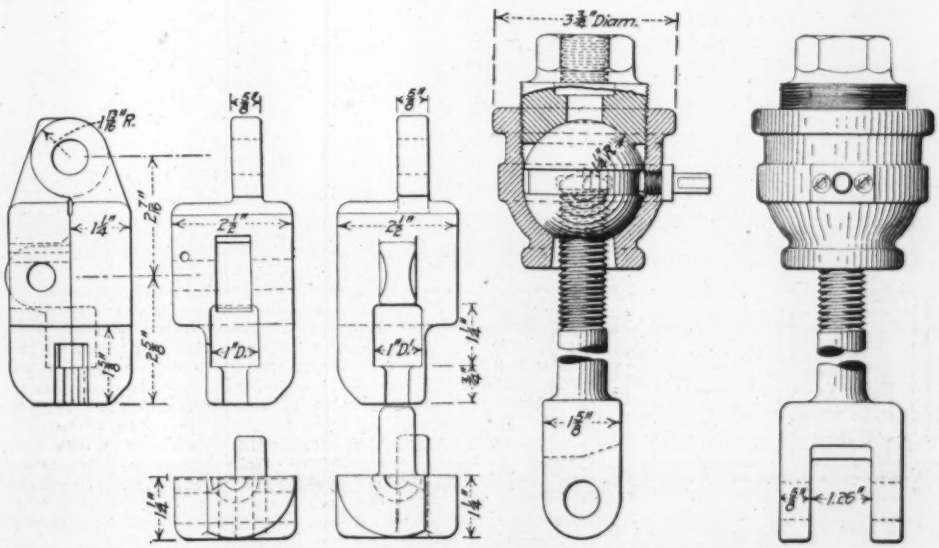


FIG. 3. DETAILS OF PENNSYLVANIA R.R. TENSION SPECIMEN GRIPS

Field and Office

Trolley-Car Concrete Mixer for Street-Railway Track

The Birmingham Railway, Light and Power Co., Birmingham, Ala., has in service two concrete mixers mounted on trolley-car trucks. They are used for practically all the inter-track paving of the company's urban car lines. One of these mixers in service is shown in the accompanying views.

A great deal of concreted track, of the type shown in Fig. 1, is being placed in Birmingham. This section com-

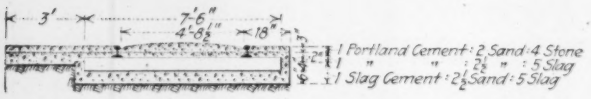


FIG. 1. TYPICAL HALF-SECTION OF CONCRETED STREET RAILWAY TRACK, BIRMINGHAM RAILWAY, LIGHT AND POWER CO.

prises a 10-in. bottom layer of 1 : 2½ : 5 concrete, in which local slag cement and a broken slag aggregate are used; a 4-in. layer of 1 : 2½ : 5 portland-cement and slag aggregate concrete, in which the top of the timber tie and the bottom of the rail are embedded; and a 3-in. finish coat of 1 : 2 : 4 portland-cement and stone concrete, crowned to the street surface.

In laying this pavement the material is brought into the street in wagons or on trolley cars and spaced in piles at intervals along the line. The mixer-car, traveling on one of the tracks to be paved, then takes the material from the piles and transforms it into the concrete to be dumped into the immediately adjacent tracks.

Each mixer-car consists of an ordinary two-truck car frame mounting a 1-yd. Smith mixer, with loading and delivery equipment. There is a regular street-car motor for both driving the mixer and propelling the car. The loading trough, Fig. 2, can be let down to the street surface and filled directly from barrows. Fig. 3 shows the mixer working along a track to be concreted and in the foreground the finish coat is being screeded.

L. L. Newman is Superintendent of Way and Structures of the railway.

Trap vs. Slag for Aggregate

Comparative tests of trap rock and slag as concrete aggregates were made during the last year for the National Slag Co., of Newark, N. J., at the department of civil engineering of Columbia University under the direction of Frederick Miller. The results for 1, 3, 6 and 12 mo. are given in the accompanying tables.

TABLE 1. ULTIMATE COMPRESSION STRENGTH OF 8x16-IN. CONCRETE CYLINDERS, IN LB. PER SQ.IN*

	28 Days	3 Mo.	6 Mo.	12 Mo.
Slag concrete	2,465	3,496	3,567	4,187
Trap rock concrete	1,975	2,961	3,053	3,537
Higher compressive strength of slag concrete	490	535	514	650
Percentage higher compressive strength of slag concrete	24.8	18.0	16.9	18.4

*The averages given are the results of testing four specimens of both types of concrete at each period.

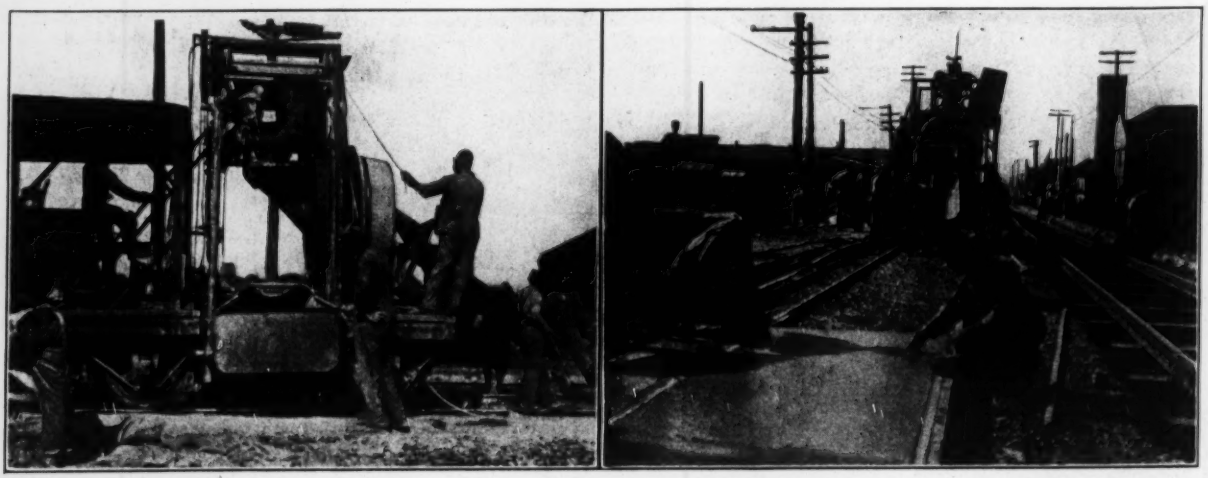
TABLE 2. MODULUS OF ELASTICITY OF SLAG AND TRAP ROCK AND TRAP ROCK CONCRETE THREE MONTHS OLD*

	Lb. per Sq.In.
Slag	4,126,000
Trap rock	4,036,000

*The specimens tested were three slag and three trap rock concrete 8x16-in. cylinders. An average value in each case was taken up to a unit load of 650 lb. per sq.in.

The average weight of slag concrete tested was 140 lb. per cu.ft., while the average weight of trap concrete was 151 lb. per cu.ft. The slag weighed 80 lb. per cu.ft., and the trap weighed 100 lb. per cu.ft.

The traprock was from the Hudson Palisades. It was separated into ¾, ½, ¼-in. sizes and "dust" and was artificially combined so that the grading was identical with that of the slag, which was used as received from the National Slag Co. The sand used was Cow Bay, a



FIGS. 2 AND 3. CONCRETE MIXER MOUNTED ON TROLLEY CAR TRUCK CONCRETING INTER-TRACK SPACE OF STREET RAILWAYS AT BIRMINGHAM, ALA.

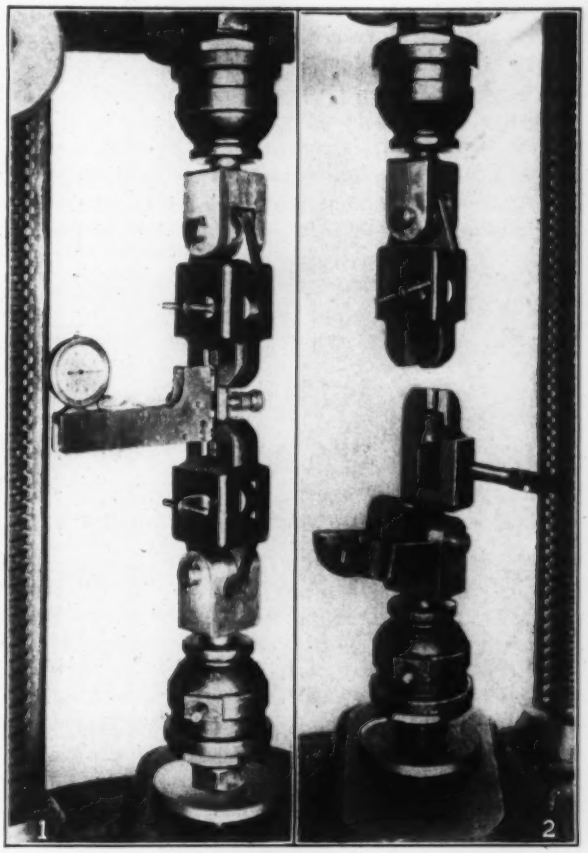
typical Long Island sand, and the cement was a mixture of three brands of portland cement, as approved by the New York Building Bureaus.

The mixture of the concrete was in all cases 1:2:4 by volume, and it was mixed in a batch mixer. In making up the complement of stone for each batch, proportional amounts of each aggregate were carefully measured by weight. It was thought that this procedure would insure greater uniformity in results.

"Dapple" Finish for Concrete

In the construction of dry dock No. 1 for the Panama Canal all the horizontal surfaces of the dock, including the coping, altars and floor, were given what is called a "dapple" finish. The method of producing this finish is described as follows by the Engineer of Terminal Construction, H. H. Rousseau, in his annual report just issued:

> The vertical surfaces of the dock were given no special treatment during placing of the concrete other than to work the mortar to the face of the form. All horizontal surfaces in the dock, including coping, altars and floor, were covered to an average depth of from 1½ to 2 in. with mortar composed of cement and specially sifted sand mixed 1:2. This mortar was applied at the same time the concrete was placed.
>
> These horizontal surfaces were all given a dapple finish in the following manner: After applying the mortar and thoroughly rodding it until the water had been worked to the surface and swept off the block with a rodding stick, the surface was thoroughly floated with a wooden float. Following this, a wooden float was used to produce the rough or dapple surface by placing the float squarely on the surface, pressing it slightly and lifting it directly upward from the surface. In this way the cement surface adhering to the float is slightly raised so as to give the appearance of chipped stone. This dapple finish is sufficiently rough to prevent slipping and makes a very suitable and satisfactory working surface.



FIGS. 1 AND 2. PENNSYLVANIA R.R. GRIPS FOR 2-IN. PLAIN SHOULDERED SPECIMEN

At left, closed, with Berry gage on specimen; at right, grips opened; ball-and-socket adjustment above and below grips

Grips for Plain-Shouldered Tension Specimens

The usual standard 2-in. test piece for forgings and castings has threaded ends that screw into the grips. This test piece requires considerably more time to make and therefore costs more than one with plain enlarged ends. Specially designed grips used in the Pennsylvania R.R. laboratory at Altoona, Penn., take such plain specimens. They require only the turning of the reduced section to proper diameter and filleting of the shoulders. Usually the ends have to be turned to get them into the space provided in the grips.

One very marked advantage of this Pennsylvania R.R. grip over others in use is that it permits the use of an extensometer or dividers on the 2-in. gage length while pulling. (One of the photographs shows a 2-in. Berry gage in place.) In other types only that part of the specimen between the jaws can be got at. A modification used by the Union Steel Casting Co. has more space inside the jaws to accommodate the enlarged section of the end of the test piece. The space will take care of a shouldered section 4 in. long. As a

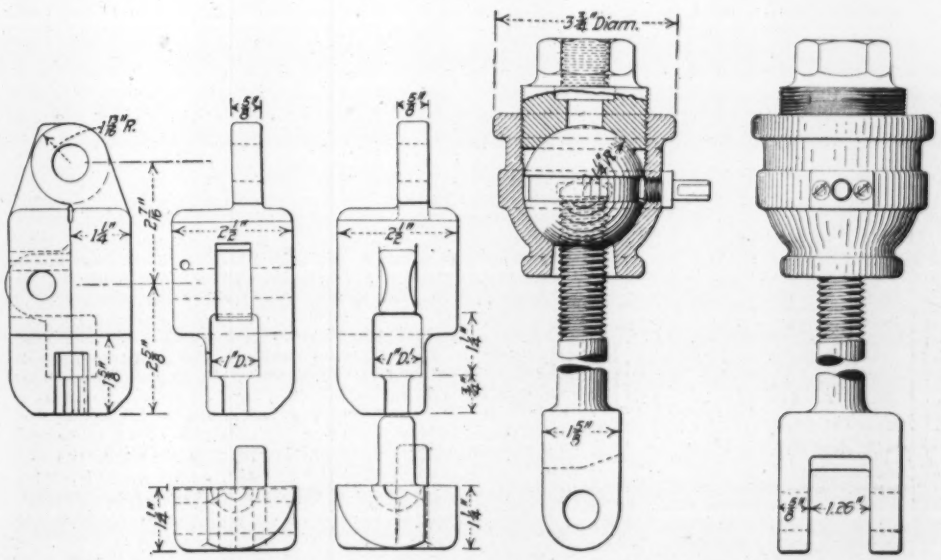


FIG. 3. DETAILS OF PENNSYLVANIA R.R. TENSION SPECIMEN GRIPS

consequence the latching arrangement for the upper grip is also modified, a yoke being substituted for the Pennsylvania R.R. tongue and pin.

The Union Steel Casting Co. uses a liberal-sized coupon attached to castings, and after this is stamped by the inspector it is removed and the center section turned down to the size required by the jaws. Saw cuts are made at the ends of the finished section, not quite

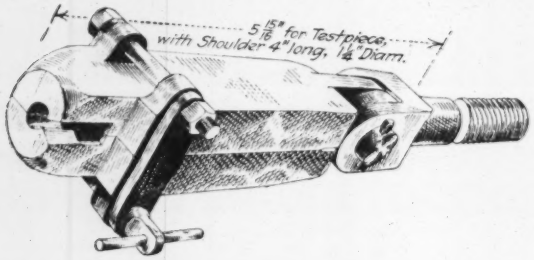


FIG. 4. MODIFIED GRIP USED BY UNION STEEL CASTING CO.

through, however, so as to leave attached the rough ends bearing the identification mark. When ready for test these ends are broken off and the specimen inserted in the grips.

The material used in these grips is hardened and tempered tool steel, which makes them immune from breakages and wear.

We are indebted to J. T. Wallis, General Superintendent of Motive Power, and C. D. Young, Engineer of Tests, Pennsylvania R.R., and to J. Lloyd Uhler, Metallurgist, Union Steel Casting Co., for the drawings and photographs.

Sinking a Shaft in Rock 180 Feet Per Month

Fast progress was made in sinking the Woodbury shaft of the Newport Mining Co., at Ironwood, Mich, last summer, according to J. M. Broan, who described the work in a paper before the Lake Superior Institute of Mining Engineers. The shaft is vertical, 13 ft. 1 in by 21 ft. 1 in. in plan, and is constructed with six compartments, as shown in the sketch immediately below.

The first 100 ft. was in quartzite; from 100 to 715 ft. there were alternate strata of gray and red slate and

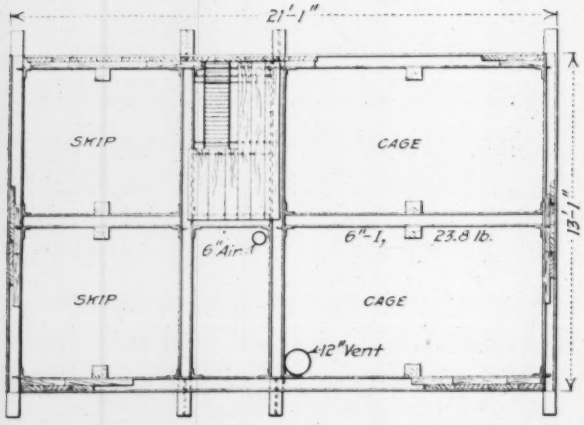


FIG. 1. PLAN OF SHAFT OF WOODBURY IRON MINE IN MICHIGAN

quartzite, below which was granite. The temporary headframe was 60 ft. high, with the trestle so arranged that it was possible to hoist rock in any of the compartments.

Excavation began on Mar. 1, 1915, and on July 27 the shaft was 901 ft. deep. The maximum monthly advance was 201 ft. and the minimum was 157 ft. Fig. 2 shows the arrangement and order of firing of holes in soft and in hard rock.

In the soft slate, hammer drills with water-tube pistons were used, and air was blown through them in place of water. In granite, where the speed was less, the water-tube pistons were replaced by the ordinary pistons. The steel was 7/8-in., made into lengths varying in 1-ft. changes from 1 to 10 ft. Until the granite was reached, a cross-bit was used, but in the hard rock excessive wear called for something more durable, and the "Carr" bit was tried out.

Considerable difficulty was encountered, especially in stratified rock, with fissured holes and stuck drills. Rather than abandon a hole, much time was spent in

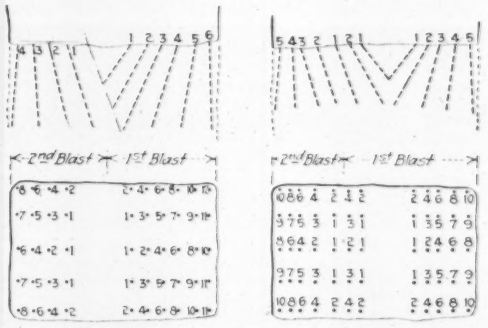


FIG. 2. ARRANGEMENT AND ORDER OF FIRING BLAST-HOLES IN SOFT SLATE AND IN GRANITE

freeing the drills. For this operation inverted hammer drills were employed very successfully.

Since the steel used was only 7/8 in. in diameter, the bottom of a deep hole was too small to contain sufficient ordinary explosive. Therefore two or three sticks of 100% blasting gelatin were placed in the bottom of each hole. The remainder of the charge, with the exception of the cartridge containing the detonator, was 80% blasting gelatin. The detonator was placed in a cartridge of 60% nitroglycerin, to speed the action of the charge. An average of 26½ lb. of explosive per foot of shaft was used in the first 800 ft., or about 1 lb. per 13 cu.ft. of excavation.

Street or Highway Grades—On the subject of establishing grades for streets or highways, T. Harold Knight, County Surveyor, Racine, Wis., has the following to say: "I believe it is customary in almost all cities, large or small, to establish the grade for streets or highways on the center line, making the height of the curb also at the established grade, allowing ¼ in. of fall per ft. across the sidewalk and parking from the property line to the curb. I suggest the following as a much better system: Establish the grade at the property line and allow the ¼ in. of fall across the walk and parking to the curb. This would make the curb grade always the same on streets with standard width of sidewalk and parking; but where this width was altered because the width of parking next to and adjoining the sidewalk was changed, the curb grade would be altered. With the old system, in cases where the parking adjoining the sidewalk is wider than standard, or where for some other reason—such as placing the extra width in a park in the center of the street—it is necessary to narrow the sidewalk parking, the resulting curb height is not at the established grade."

Laying a Submerged 8-In. Pipe Sewer on a Pile Trestle

In a paper read before the Boston Society of Civil Engineers, Dec. 13, 1915, Charles R. Gow, contractor, of Boston, Mass., described the laying of an 8-in. cast-iron pipe across a narrow stream, to form a part of a sewer, so that the pipe had to be laid to an established grade. What follows is substantially in Mr. Gow's words.

The stream was approximately 75 ft. wide and about 5 ft. deep. As the river was used for boating, no obstruction was permitted. The bed of the stream was gravel overlying a deposit of mud. The crossing was located close to and parallel with an existing highway bridge, from which all of the work was carried on.

A steam derrick mounted upon a platform with rollers was erected on the bridge, so that it could be moved back and forth as the work progressed, the boom being long enough to reach out over the pipe location. A small orange-peel bucket was used to remove the gravel slopes at either bank as well as to dredge the shallow trench necessary for the pipe. A pair of wooden leads containing a piledriving hammer were suspended from the boom end. The piledriver was securely guyed in position, while spruce piles were driven in bents of two, spaced 3 ft. on centers. There were 22 piles or 11 bents.

It was necessary to cut and cap the piles at exact grade, and as the cutting point was more than 5 ft. under water the following method was devised to accomplish the purpose:

A light riveted steel cylinder 6 ft. in diameter and 10 ft. in length was lowered over a bent of two piles until it rested vertically upon the bottom. To seal the bottom edge, some sand and gravel were deposited around the outside. A pulsometer pump was then swung out by the derrick, the suction lowered into the cylinder, and by means of a steam connection with the hoisting engine the water was removed from the interior of the cylinder so that men could enter, cut the piles with an ordinary crosscut saw and cap them at the required grade with 6x6-in. timbers. When this operation was completed, the cylinder was withdrawn and placed over the next bent of uncut piles.

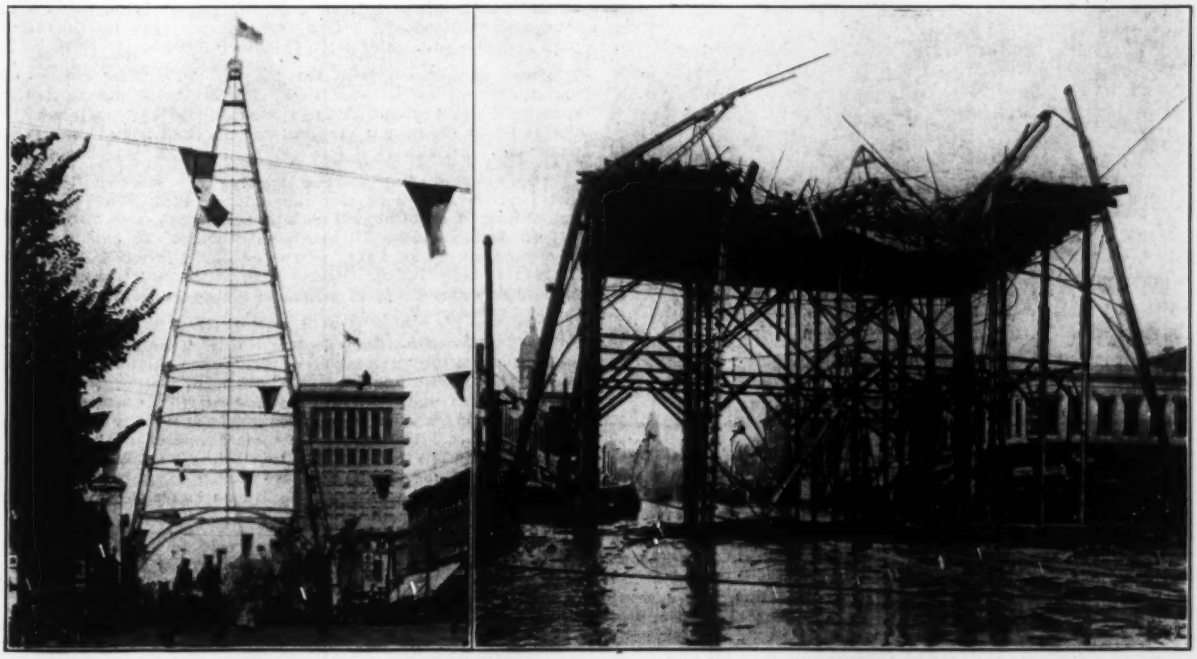
When all of the pile bents had been thus capped, the pipe was assembled on the bridge, the joints leaded and calked, and by means of the derrick and a guyed gin pole the entire length of pipe was suspended over the desired location and lowered through the water till it rested on the pile caps. A diver was employed to wedge the pipe securely in place, this being the only part of the work on which a diver was used. The pipe was then backfilled with some of the dredged material.

The entire cost of this work exclusive of the cast-iron pipe and lead for joints was $561.

Steel Lighting Tower Fails After 34-Yr. Service

A steel electric-light tower was blown down in a windstorm in San José, Calif., on Dec. 3, 1915. The tower (Fig. 1), erected in 1881 at a cost of $3,465, was being repaired at the time of the collapse, and the wreckage fell on the scaffolding and on the street. The height was 199 ft.; the base was 75 ft. on each side. The lower part of the four columns was built up of two 4-in. pipes, while the horizontal and sway bracing also were made of pipe, most of the sway-bracing being curved. The bracing rods were not upset.

The tower gave no appreciable sign of deterioration until a year ago, when it was damaged to some extent by a wind storm. It was then proposed that the structure be reinforced, and a design was prepared by L. T. Alden and A. A. Pyle, of the engineering department of the State Harbor Commission, and G. J. Adams, of the



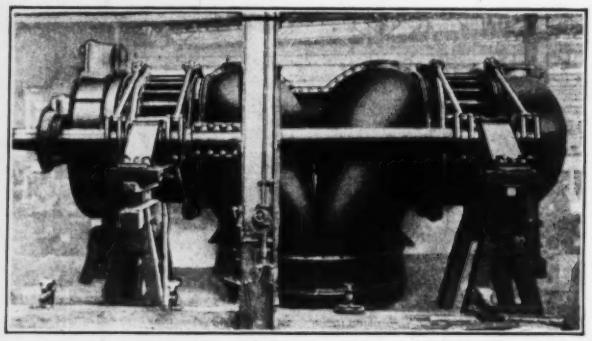
FIGS. 1 AND 2. TOWER AS IT LOOKED BEFORE FAILURE AND AS IT APPEARED AFTER COLLAPSING ON FALSEWORK DURING REPAIR

State Architect's office. The design followed the general plan of the old structure, most of which was to be retained. Steel angles were to be added throughout. The contract for the work was awarded to the Golden Gate Iron Works, of San Francisco, for about $6,000. At the time of the storm about 50 ft. of the new structure had been erected. After the accident an investigating committee found that the contractor was not at fault, as the contract held him responsible only for failures resulting from poor workmanship. It is proposed to complete the tower in spite of the failure, and for that purpose $2,500 will be raised in addition to the sum already on hand.

Double-Runner Turbine Shows 88% Efficiency

Single-runner hydraulic turbines of about 90% efficiency have become fairly common, but power-plant designers do not generally look for results of this order in double-runner machines. Therefore the three double-runner machines placed in service in 1915 in the Point du Bois hydro-electric station of the City of Winnipeg are of interest, as an efficiency figure of 88% is claimed by the builders—Escher, Wyss & Co., of Zurich, Switzerland.

The wheels were specified to develop 5,200 hp., and penalties were imposed for failure to reach rated capacity, varying from $500 for 100 hp. to $10,000 for 800 hp. In three tests at 80% gate opening and 46-ft. head, figures of 7,020, 7,155 and 7,220 b.hp. were obtained. The company guaranteed 6,800 hp. under 45 ft. and at 84% efficiency. The claimed 4% gain in efficiency performance was deduced from the facts that the draft tube was the limiting feature of the turbine installation and that the power of the runners fell off when the gates were opened more than 80%.

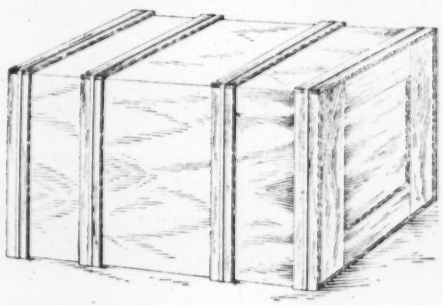


WINNIPEG 7,200-HP. DOUBLE RUNNER

NOTES

Relieving Traffic Congestion in New York City—The traffic congestion at 42nd St. and Fifth Ave., New York City, and at several other important corners has been a subject of study by organizations and individuals for some time, and several schemes have been put forward for relieving it. Anthony J. Daino, of 317 W. 80th St., New York City, offers a scheme for partly relieving the congestion here and at 34th St. and Fifth Ave. It is suggested that a bridge 8 ft. wide be erected on each side of Fifth Ave. for pasenger automobiles, leaving a 24-ft. roadway for busses, trucks and parades. These bridges would insure the continual movement of a single line of automobiles in each direction. Of course alternate stoppages of general traffic on the street level would still be necessary, but a large part of the automobile traffic, which is the greatest problem at this corner, would not have to be held up on Fifth Ave. to allow the movement of crosstown traffic. Tunnels could be provided under 42nd St. so that pedestrians could proceed without regard to travel on Fifth Ave. This plan would partly relieve conditions at this congested point, but whether sufficiently to warrant the building of such structures on this important corner is a matter for expert consideration. Certainly the bridges should be designed to present as pleasing an appearance as possible. Mr. Daino recognizes the imperfections of his suggestion, but believes it to be the simplest plan for partial relief yet proposed. He has submitted his drawing to the Police Department.

A Satisfactory Packing Case for the South American trade has been brought to the attention of L. J. Keena, United States consul general at Valparaiso, Chile. The complaint of poor packing of goods has been made against American manufacturers for many years, so that a really satisfactory box ought to be gazed upon with avidity and copied shamelessly if the cost of the box be not too high. It will be observed that the end blocking and the end battens entirely cover the joinings of the side with the top and bottom of the case and



A PACKING CASE WHICH A SOUTH AMERICAN IMPORTER RECOMMENDS

Dimensions: 3 ft. 4 in. by 2 ft. 4 in. by 2 ft. 4 in. Zinc lined, nailed, battened and hooped

give rigidity. The case illustrated is zinc lined and is 3 ft. 4 in. long by 2 ft. 4 in. in height and width. The ends are 1¼-in. boards, the sides 1 in., and the battens are made of strips ¾x3½ in. It is strongly iron-hooped around the battens and well nailed. For a case of this type the importer pays an extra charge of $6 to $7.25 to the exporter.

Rails Weighing 100 to 141 Lb. per yd. are in service on about 1,000 mi. of the New York, New Haven & Hartford R.R. system; about 2,300 mi. of the system is laid with rails weighing 78 lb. to 100 lb. per yd., and on less than 200 mi. are rails weighing less than 70 lb.

The Cost of Engineering Service for construction and maintenance involving the expenditure of $3,334,574 on the public works of Minneapolis, Minn., in 1915, was $73,000, or 2.18%. This includes all overhead charges. In 1914 the cost of engineering was 2.27% of the cost of the work done, and in 1913 it was 2.32%. Prospective municipal work in 1916 amounts to $3,405,000, of which the biggest item is for paving, $1,250,000.

Road Shoulders Rolled by Motor Truck—An original method of rolling the shoulders of a concrete-paved road was used by a contractor in Santa Barbara, Calif. He took a 5-ton motor truck with a body for carrying earth and removed one of the rear wheels, replacing it with a steel wheel, or roller, 18 in. wide and of the same diameter as the opposite drive-wheel. The body of the truck was kept loaded. With this device he rolled the earth shoulders for a width of 4 ft. on each side of the pavement quite as effectively as the use of a regular road roller.

Large Blast in Norway—A blast said to be the largest in Europe (for purposes of peace) was set off last summer in the north part of Norway. The iron-mining company Südvaranger blew off the top of a mountain and its face, using 23,000 kg. (25 tons) of dynamite. The rock and ore brought down are estimated at 300,000 tons. The large-blast system was initiated by the company in 1911, with a shot of 8,000 kg. Subsequent blasts used dynamite in quantities of 10,000, 12,000, 14,000, 18,000 and 23,000 kg.—"Stahl und Eisen."

Editorials

Compression Tests of Steel for Compression Service

Because tension tests are convenient, we test steel in tension only. Whatever the intended service—tensile, compressive, torsional, or plain shearing—the tension test is the criterion of acceptance. Rivet steel, for example, is supplied and accepted only on the basis of tensile strength and ductility, with possibly a nicked-bend test added.

Every day hundreds of columns and struts, having nothing to do with tension, are made from material whose suitability is judged by nothing but tension tests; and in our modern life it may be said that the whole world is carried on steel columns. The condition is based on long-standing tradition, but it is not logical.

Concrete men, younger in their practice than the workers in steel, have not followed the same method. They have always made their quality tests of concrete by compression; and now that acceptance tests of concrete are coming into use, they employ a compressive test here also. All this is obviously based on the natural view that compressive service calls for testing in compression. These same concrete men have hitherto been inconsistent to the extent of judging their chief raw material, the cement, by a tension test. But now a strong movement is on foot to do away with the tension test of cement and use the compression test instead; if we judge rightly, it may be said that the cement men are on the verge of adopting the compression test as standard. They are ready to break with traditional custom in order to meet the logical claim of compression tests for compression service. The analogy has some force for structural steel.

A striking fact was brought out recently in the first published results of the great series of column tests made by the Bureau of Standards for the American Society of Civil Engineers. The figures showed that certain columns were weaker than others, and the reason was not at first discernible, as these weaker columns had the higher tensile yield point; but on further investigation it was found that in compression test their yield point was low. It is obvious that low elastic properties in compression will produce weak columns. The striking feature of the case is not that these columns were weak, or why they were weak, but the fact that in practical work we would have trusted them unsuspectingly with more load than they should be allowed to carry. A danger was revealed that had not been recognized before. Lack of compression tests created the danger.

It is not profitable to inquire whether many existing columns contain the weakness disclosed by the tests mentioned. The inquiry would be mere speculation, because not enough experience with compression tests of the currently used grades of structural steel is available. The profitable, the necessary, thing is to find out whether our future columns contain this weakness, or rather to make sure that they do not. For this purpose the steel from which they are to be made should be tested in compression.

The specimen and the machine manipulation for a compressive test of steel are much simpler than those required for a tensile test—a favorable argument. On the other hand, the matter of suitable methods and instruments for determining the yield point, or perhaps a more elusive elastic limit, are not so simple. Some clear-headed experimenting may have to be done to reduce the methods to most convenient form for practice. There will also be occasion for deciding how well the "commercial-laboratory" methods of steel testing are suited to making reliable compression tests. The net result, however, will probably be that such tests will prove to be no more difficult, time consuming or costly than the tension tests now standard.

Testing material by a new standard tends to produce better material—better by that standard. The constant use of the tension test has given us a steel whose tensile qualities are excellent—indeed, even improving. It is certain that the use of a compression test would improve the compressive resistance of steel; and there is no reason for thinking that this would involve sacrifices in other desirable qualities.

The logical necessity for testing compression material by compressive test has been disregarded hitherto by force of tradition. Now that the column tests have shown a danger in following this tradition, the time is ripe for reconsidering the practice of testing column steel in tension only.

Why Pick Out the Bone Patent?

There must be now upward of 1,000 United States patents on reinforced-concrete structures. Why should an engineering journal pick out from this great number one lone patent and honor it with so extended a discussion as the one which appears on other pages of this issue?

In answering this question the editors may cite, first, the controlling consideration in all journalistic work; that is, the number of readers interested. Probably no one structural development has had so wide adoption as the reinforced-concrete retaining wall. Its numbers are legion; its use, universal. And thanks to Frank A. Bone's efficient detective methods, if his rights to a royalty on all such walls are conclusively established there will hardly be a city or a railway company in the United States that will not have to pay tribute to him.

But even the great interest in the patent or the large number of engineers affected by its enforcement would not have been sufficient to warrant extended discussion, had it not been of such relative simplicity and anticipations of its general principle so readily obtained. Unlike a number of the famous reinforced-concrete patents, that issued to Mr. Bone is upon a simple, plain structure the general features of which are easily understood and the complications in which can only be in minor de-

tails. The claims of the patent, while characteristically involved and extended, nevertheless reduce to terms descriptive of a simple structure, in which respect they differ from most structural patents. The space and time required to analyze the patent could therefore be reduced to such an amount as is permissible in a periodical publication.

Finally, the available literature of anticipation is so rich and the disregard of that literature by the defendants in the earlier Bone infringement suits so conspicuous that justice to the engineering profession and the public demands that the full facts in the case be set forth.

The facts recited in this issue will serve to confirm the belief generally held by engineers that cantilever reinforced-concrete retaining walls were not new in 1898 and that their design at that time did not require invention, but only the application of well-known mechanical principles to a well-known type of structure. The attorneys and experts who represented Marion County at Indianapolis are to be congratulated on the clear way in which they presented their case and Judge Anderson is to be commended for his ready appreciation of the engineering facts they laid before him.

As for the cantilever counterforted wall, which Mr. Bone once included in his demand for royalty, but which more recently has been repudiated as an infringement, it is evident that not only was it absent from Mr. Bone's 1,899 application, but furthermore that it is of at least equal age and authority as the pure skeleton wall.

Engineers or Manufacturers as Designers of Machinery

In *Engineering News* of Jan. 13 comment was made on the unusual procedure adopted by the New Orleans Sewerage and Water Board in making complete designs and specifications for the new screw pumps for handling the city drainage instead of relying on manufacturers to furnish designs. The comment was chiefly directed to showing that the results obtained by the new pumps in economy and efficiency were so important and resulted in such a large saving in annual operating cost that this unusual procedure was justified.

Since that editorial was printed, additional information has been received, showing that the Sewerage and Water Board has not merely secured much better pumps than it could have bought in the open market, but that there was a large saving in the first cost of the pumps by having the work done in this way.

Those who have followed the history of the drainage work in New Orleans are aware that it differs materially from any work of a similar sort carried on elsewhere. The experience of a dozen years in the operation of the New Orleans drainage system has developed the great advantage of certain special characteristics in the pumping machinery. The engineers in charge of the work, therefore, were very much better able to specify the conditions that a pumping plant should fulfill than were the engineers of any manufacturing concern.

The engineers of the Sewerage and Water Board might of course have adopted the plan often followed of drawing a general specification for the conditions which the machinery should fulfill and inviting bids on machinery designed for these conditions, with requirements as to material and workmanship and premiums for securing or exceeding the desired efficiency. There are, however, very few manufacturing concerns in the country with a technical staff competent to deal with such a problem and with the necessary time to devote to it apart from their regular work. It was foreseen, therefore, that if bids on such a basis were invited, only two or three proposals were likely to be received, and these would naturally include a large amount for the cost of the original development work and the risk of failure to meet the specified requirements. Further, it was not unlikely that if the responsibility of design were thus left with the manufacturers, some features might be included that the New Orleans engineers would regard as objectionable. Yet they could not insist on their omission, without in part, at least, relieving the contractor from the responsibility for success.

In the plan adopted, under which the engineering staff of the Sewerage and Water Board has itself prepared the complete designs and specifications, any first-class machine shop capable of handling large-size work could put in a bid with the element of risk practically eliminated. In this way sharp competition was insured; and when the bids were opened, it was found that eleven proposals had been received, the prices ranging from $159,000 to $446,000.

It should be understood that while the screw pump adopted at New Orleans is original in design, its success was demonstrated before it was adopted for this large plant. There was first built in the city's own repair shops a 12-in. experimental pump at a cost of about $2,500. After this was demonstrated to be a success, a 30-in. pump was built that cost, including all expenditure on tests and some changes, about $4,000. This pump will be used in regular service; and its total cost is only about $1,000 in excess of pumps of the same size built by contract since, so that the total spent for experimental development, etc., chargeable to the entire plant, is about $3,500.

It is true that there was a certain outlay for the preparation of plans, specifications, supervision of construction, etc., but in comparing these items of cost with the expenditure on previous contracts let by the board for machinery, where considerable experimenting had to be done to develop tests and conditions that a contractor was willing to accept, with the extra expenditure on tests made to prove a contract performance, it is concluded that the plan adopted in the present case has been no more expensive than the former plan in direct outlay and has been far superior, as has been already shown, in the efficient results obtained.

The cost of the New Orleans pumping plant per cubic foot per second of capacity demonstrates the large saving made in the first cost of the new plant by using the engineers' designs instead of attempting to use manufacturers' standard equipment, as was done in the original installation at New Orleans. The actual cost per cubic foot per second of capacity of the new plant, including pumps, shafts, bearings, suction and discharge pipe and priming apparatus, but excluding foundations, buildings, suction and discharge basins and motors, is about $30. The original equipment of standard centrifugal pumps at New Orleans, which was purchased under competitive bids and contracts, including the same items as those specified above, cost more than three times as much, the contract prices ranging from $90 to $110 per cu.ft. per sec.

capacity. In using these figures it must be understood, of course, in each case that a permanent plant was erected, designed for reliability and high efficiency, and the figures cannot be compared with the cost of ordinary high-speed centrifugal pumping equipment of small size, built for special uses and with low first cost the main idea.

It is especially worth while to comment on these results obtained at New Orleans, because there is always a strong tendency on the part of those who hold the purse strings to trust the manufacturer rather than the engineer in problems of technical design. It is true, of course, of very many lines of machinery that an article of standard manufacture represents the best that can be obtained; but when a machine is desired to fill unusual requirements, it is generally found that a manufacturing concern will not take up the design and give the necessary study and investigation to produce the best results. It is more and more the custom, in large companies operating power-plant and other machinery, for the company's engineers, who have come to understand the requirements that the machinery should fulfill and who are more familiar with the defects that develop in operation than are the manufacturer's engineers, to specify exactly what shall be furnished and in many cases to work out designs to fulfill the requirements.

Is the Rock-Fill Dam a Safe Type of Dam?

The destruction of the Lower Otay rock-fill dam, near San Diego, by the flood of Jan. 27 will almost inevitably have an unfortunate effect in causing the public to distrust the safety of dams and particularly to be suspicious of dams of the rock-fill type. It could not well be otherwise. In the literature published by the company that built the dam and later sold it to the City of San Diego the structure was praised as a marvel of engineering skill. So far as the public was aware, the Lower Otay dam was as substantial and permanent a structure as was ever built.

In the engineering profession, however, the Lower Otay dam was not held in such high repute. In very few places has its peculiar design been imitated. It seems important to call especial attention to the causes of failure of the Otay dam, for when clearly understood, it will be realized that the disaster carries no warning as to lack of safety of dams in general or of the rock-fill dam as a type. The failure does make evident that a rock-fill dam built as was the Otay dam is not materially safer, if overtopped, than an earth dam.

The defects in the Otay dam that brought about the destruction appear to have been, first, too small a cross-section for a rock-fill structure; second, failure to pave the downstream slope of the dam so that it could withstand the action of water flowing down it; and, third, lack of provision of a substantial crest to the dam capable of standing the overflow of water.

It would have added a very small percentage to the cost of the dam to have provided all these things. The solid rock sides of the cañon in which the dam was built towered high above it. It would have been a comparatively small expense to have filled in the downstream side of the dam and have given it a flatter slope. Then this slope could have been covered with stone blocks weighing from half a ton to two or three tons, set close together and chinked with small stone.

The laying of such a paving on the downstream slope would have cost nothing but the blasting of the stone and its placing. An even crest to the dam would have been furnished if the concrete core wall with its steel embedded plate had been carried up to the top of the dam instead of stopping some 5 ft. below.

It may be asked why the dam was not built in this way in the first place. The answer is that, when the dam was built, the company which owned it was chiefly anxious to do the work at the least possible cost. The company and its successors have always worried far more over lack of rainfall to fill the dam than over floods that might overtop it. Nor is this strange, when one reflects that never in the 19 years since the dam was built has the reservoir behind it been filled until the storms of last month came.

There is one other matter which should be given attention in the design of a rock-fill dam, if it is to pass a sheet of water over its crest with safety. Care must be taken in depositing the rock that too much earth and fine material are not placed with the rock.

In the Lower Otay dam, we are informed, there was so much earth mixed with the rock on the downstream side that probably the whole mass settled when the flood passed over the crest. Even if the lower slope of a rock-fill dam is paved, a large amount of water will percolate through the mass and wash out any earth in it. There must be solid rock enough in the mass so that the dirt can wash out without affecting the stability, and the rock must be in large enough masses so that the water can escape freely at the lower toe without setting up hydraulic pressure. All these things are perfectly easy to do, as a rule, in locations where the rock-fill dam is the type best adapted to conditions and can usually be done at less cost than widening the spillway enough to carry any possible flood.

Should the Fahrenheit Scale Be Abolished?

A bill has been introduced in Congress to abolish the Fahrenheit thermometer scale and require the use of the Centigrade scale instead in all United States Government publications. Circulars to test public opinion on such a proposed change have been sent out by the local Weather Bureau offices. The circulars contain the suggestion that if a change in the standard scale were to be made, some other scale than the Centigrade might better serve the purpose, as the use of the Centigrade scale requires many minus signs.

Opinions of engineers would doubtless be divided as to the wisdom of abolishing by law the use of the Fahrenheit thermometer scale. A very large proportion of the profession is accustomed to the use of the Centigrade scale in calculations and would doubtless approve the change, but many engineers have never learned to think in Centigrade terms. Much of the Government work in its scientific departments is already done with the Centigrade scale. The proposed law if enacted would have its chief effect in the Weather Bureau, which would be obliged to use the Centigrade scale exclusively in its weather reports. In the country districts particularly, where there is especial interest in the weather bulletins, it is doubtful whether a change to a new-fangled thermometer scale would meet with favor.

Letters to the Editor

"Stress or Strain"

Sir—There are many engineers still in practice who remember when the word "strain" was in universal use to signify both applied strain and internal strain. Then came the time when, for the purpose of the classroom, it seemed advisable to make a distinction between applied strain, or pressure, and internal strain, or deformation; and the word "stress" was introduced for applied strain, with the apparent thought that we were following such an authority as Professor Rankine.

As a matter of fact, it appears that Professor Rankine used the word "stress," not as meaning applied pressure or external force, but rather as the internal force which a molecule under deformation exerts in any direction tending to recover its free condition and which for a state of equilibrium is equal to the strain that produces it. His use of the word "stress" in this sense was well advised and corresponds to the internal stress that an individual may feel from various causes. Stress is, properly speaking, the result of strain, and not strain the result of stress.

"Stress" and "strain," in general language, appear to have much the same meaning, except that "stress" is a refinement. All pressures in bridgework are large and forceful and are, properly speaking, not merely stresses, but are actual strains, whether they be external or internal.

The earlier use of the word "strain" in its double sense was not considered sufficiently discriminating, but the present use of the word "stress" has become more indiscriminate and incorrect than the use of "strain" ever was, for the latter was always considered in its double application. When we see the expression "the elastic-limit stress," when the elastic limit is purely a physical condition or strain and results at once in a momentary decrease of stress; or when we hear the term "secondary stresses," when they are never secondary "stresses" in the modern sense, but are always "strains," both in origin and in effect; and finally when we hear the very common expression "the material is overstressed"—then we realize that our "reform" is merely an æsthetic effort, psychological rather than fundamental, and is unfortunate and incorrect.

We have for years abandoned the good old term "strain-sheet" for the more æsthetic "stress-sheet," with its triple "s", and in order to be consistent may yet call it a "stress-and-strain-sheet," since from the so-called "stresses" we deduce the strains to which the material is subjected and proportion our bridges accordingly.

Should we not leave the "æsthetic" for the ornamentation of the structure rather than introduce it into the discussion of the mechanical design? My own preference is to abandon the word "stress" and to use the word "strain" as formerly. Such was also the preference of our friend the late C. C. Schneider.

HENRY B. SEAMAN.

Woolworth Building, New York City, Jan. 21, 1916.

Value of Co-operative Schools

Sir—The plan of the Mechanics Institute, of Rochester, N. Y., for combining practice with theory, as described in *Engineering News*, Jan. 13, 1915, is no doubt an earnest endeavor to broaden the experience and mental vocabulary of the young technical student. It is avowedly designed to make him immediately "useful to his employer." Although the writer has no knowledge of the details of the system, it is apparently the result of a narrow-minded movement that would stifle the spirit of engineering. Experience is vital; ambition and daring are the life-blood of the profession, but science is its brain.

When the student leaves college and enters the busy world of construction, he seems to plunge from an atmosphere of exact science into a cold bath of practicality, where apparently all his treasured mathematics is smothered in empiricism and approximation, where men bow down and worship experience. Practical experience comes to him easily enough. But he finds slight incentive to use his mathematics. Now comes the test. Unless his theory has been so pounded into him that it has become the very essence of his mental life and unless he has enough ambition and spirit to pursue that which he has only glimpsed, theory gradually slips away, and he becomes a mere technical mechanic, seeing no farther than the end of his nose—or what is the same thing, no farther than his handbook.

By all means let us have experience; but when the young engineer is in college, let us endeavor so to ground him in the fundamentals and traditions of the profession he aspires to enter that he will become an active member of it, a producer—not a parasite on other men's gray matter. Practice will come from without, unasked; the development of theory can only come from within.

STEWART C. BARNETT.

1617 Oxford St., Philadelphia, Penn., Jan. 16, 1916.

Is a Part Stronger Than the Whole?

Sir—In *Engineering News* of Nov. 25, 1915, R. Fleming discusses the impossibility of always applying the accepted rules for eccentric stresses in practical construction.

The writer is familiar with "Case 3," relative to the 50-ft. coffer-dam wall, 6 ft. thick, which was loaded at the outer third-point by a line of wall columns. This design was approved by the Bureau of Buildings, inasmuch as the eccentric stress due to vertical loading was sufficiently neutralized by the lateral pressure on the wall due to earth and water at this great depth. The existence of hydraulic pressure was signally demonstrated by a sliding of the wall previous to the insertion of lateral braces (eventually replaced by basement and cellar-floor systems). The foundation as constructed employed cantilever foundation girders anchored to interior columns.

The question of eccentric stress is a continually recurring one in engineering problems. A design frequently met with in building construction is shown in Fig. 1. To provide for the distribution of two wall-column loads, applied adjacent to the property lines of the building lot, cantilever girders parallel to the lot line are employed. The foundation wall is carried to rock and made continuous under the three distributing girders. The required section modulus to distribute the load uniformly is 530,

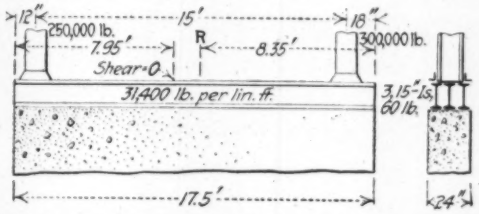


FIG. 1. FOUNDATION GIRDERS 130% OVERLOADED IF PRESSURE IS UNIFORMLY DISTRIBUTED

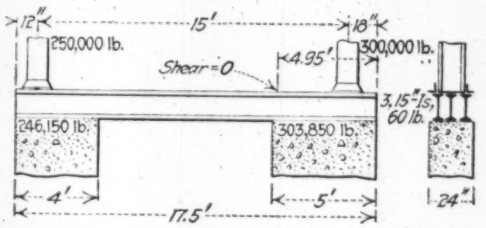


FIG. 2. SAME GIRDERS MADE SAFE BY CUTTING OUT MIDDLE OF PIER

whereas the girders possess a combined modulus of only 244. The girders should therefore be 130% overloaded.

If, however, the middle portion of the foundation wall is removed, as shown in Fig. 2, the same girders possess an excess strength of 11%, a section modulus of 220 being required. Will the girders shown in Fig. 1 deflect just sufficiently to give a distribution of the loads at both ends of the wall approximating that shown in Fig. 2, or is a part stronger than the whole?

New York City, Jan. 3, 1916. GEO. E. STREHAN,
Assistant Engineer, Bureau of Buildings.

Possible Co-operation Between Engineer and Architect

Sir—In your issue of Jan. 6, 1916, appears an editorial on "Engineers and Bridge Art." It is very evident that the writer of the editorial in question does not approve of the decorative features of the Q St. bridge at Washington, for he speaks of it as "almost tawdry."

In the *American Architect* of Oct. 27, 1915, appears an article on the same bridge. In this article the bridge is referred to as "a notable achievement" and as "an example of the conjunction of good architecture and good engineering . . . worthy of extended comment and illustration." The statement is made, "If the result achieved in this instance is carefully considered in future similar undertakings, we may hope with considerable certainty for the development of a closer relationship between architectural and engineering problems."

The writer is a bridge engineer connected in a responsible way with the design of large and important structures and, so far as his opportunities permit, is an exponent of this closer relationship between architectural and engineering problems.

The whole subject of artistic bridge design is one of great difficulty and of many vexations, and it seems to the writer as if such a paper as yours should be strong in its leadership and accurate in its judgment concerning this matter. The same statement applied to the *American Architect* as the exponent of architecture.

If, however, such representative opinions as these two honest and carefully considered editorials differ so radically, how can there be an agreement between the professions at large?

The remarks already made prompt another. It is at first almost impossible for the average architect to cooperate with an engineer. The engineer is bound by certain considerations of cost or strength, which must be accepted by the architect if he is to help. These limitations are often so irritating to him that he is unable or unwilling to accept them gracefully. It sometimes happens that the engineer, bound by tradition and practice, lacks the authority to break entirely away from precedent. Under such conditions coöperation becomes very unsatisfactory. Many times the writer has appealed to his architectural friends for aid in the proportioning of a pier cap or some other detail, and he has seldom had the nerve to propose the results to his chief.

This whole matter is of growing importance, and it is to be hoped that those who lead professional opinion will take it up in a broad and sympathetic way for the benefit of us younger men who need the help in our efforts to combine beauty with strength and economy.

JOHN F. JOHNSTON.

Sacramento, Calif., Jan. 20, 1916.

NOTES AND QUERIES

Hatchtown Dam Failure—In the article "Analysis of the Failure of an Earth-Fill Dam," in the issue of Jan. 13, 1916, p. 56, the captions of Figs. 6 and 7 were not correct. They should have read: "Fig. 6. Old Culvert as Exposed by the Failure. Fig. 7. Downstream Part of New Culvert Before Construction of Fill."

Weather Bureau Wants Data on Earthquakes—In the issue of Dec. 23, in commenting on the recent earthquake in northern Nevada, allusion was made to the desirability of having data regarding the occurrence of earthquakes collected by some Government department. It is now learned that the United States Weather Bureau has undertaken the systematic collection of earthquake data and has blanks on file at all its stations for reports of any earthquake experienced by the observer. The Bureau will be very glad to have the coöperation of engineers, especially those located in remote sections of the country, in this collection of earthquake data. It will be glad to send to any engineer, on request, samples of the cards used in reporting earthquakes.

Strengthening of New York Highway Bridges that are unsafe for heavy loads is under consideration by a committee of the legislature. The committee was created by a joint resolution of the legislature Apr. 23, 1915, to inquire into the condition of all bridges in New York outside of cities and the methods in use for maintenance, and to propose the development of a uniform system of construction and maintenance of bridges, culverts and sluices in the state. The committee is to make its report to the legislature by Feb. 15. According to a public statement by the chairman of the committee, Charles O. Pratt, there are about 34,000 highway bridges in New York State which are maintained by the townships, and it is probable that at least 10,000 of these bridges are unsafe to carry a rolling load as great as 5 tons. The committee is considering a recommendaion to the legislature that these bridges shall be made safe for loads of 15 tons. It is estimated that the cost of rebuilding all the bridges in the state to this standard would be as much as $30,000,000. Great as this sum is, there must be considered with it the claims against the state for damages resulting from accidents on unsafe bridges. There are now before the State Court of Claims more than 100 cases for damages from bridge failures.

Father of Waters in Flood

By Major E. M. Markham*

SYNOPSIS—Successive rainfalls since the middle of December in the various tributary drainage areas brought both the Ohio and Mississippi Rivers into flood at their junction and gave rise to what promised to be almost, if not quite, a record flood stage below Cairo. These conditions and their effect north of the point where the Arkansas and White Rivers join with the Mississippi are reviewed up to Feb. 5.

The foundation of the present flood situation in the Mississippi River north of its junction with the Arkansas and the White was laid as far back as Dec. 16 in the development of an area of low barometer that moved northwest from Texas over the Central Valley, thence up the Ohio and across the Middle Atlantic States, passing into the Atlantic Ocean on Dec. 18.

Precipitation on Dec. 16 was not excessive, there having been 0.2 in. of rain at Little Rock; 1 in. at Fort Smith, Ark.; 0.78 at Louisville, Ky.; 0.48 at Cairo, Ill.; 0.52 at Springfield, Mo.; and 0.32 at St. Louis. On Dec. 17, however, precipitation at Little Rock had increased to 1.6 in.; Cairo, Ill., 1.74; Evansville, Ind., 2.76; Louisville, Ky., 4.18; and Lexington, Ky., 2.04. On Dec. 18 heavy rainfall continued over the Tennessee and Cumberland Valleys, as indicated by 1.94 in. at Nashville, 2.6 at Chattanooga, 2.32 at Knoxville and 1.3 at Lexington. Precipitation was also general over the Gulf and Atlantic States and as far west as a line roughly from Detroit, Mich., to Little Rock, Ark. This storm had little effect upon the upper Mississippi, but its influence on the Ohio was marked by an increase of stage at Cairo from 17.9 ft. on Dec. 16 to 42 ft. on Dec. 30.

A Second Area of Precipitation

On Dec. 28 and 29 a second area of precipitation, partly rain and partly snow, extended from Texas in a generally northeasterly direction over the lower Mississippi and the entire Ohio River Valley. This low area passed into the Atlantic on Dec. 30, after general, though mild, rainfall, except along the Ohio and its tributaries, a precipitation of 3.2 in. at Chattanooga, 2.5 at Knoxville and 1.14 at Cairo, Ill., being recorded.

This second storm carried the Cairo gage past its flood stage of 45 ft. to a maximum of 49.5 ft. on Jan. 17. In the meantime light rains had again developed over the entire Ohio River watershed on Jan. 6, 7 and 11. On Jan. 12 a storm area of considerable intensity centered over eastern Missouri and central Illinois, resulting in a rainfall of 2.62 in. at Springfield, Mo.; 2.96 at St. Louis; 1.46 at Springfield, Ill.; 1.54 at Peoria; 1.12 at Louisville, Ky.; 1 at Cincinnati; and 1.2 at Parkersburg. Scattered precipitation was also general, though nowhere excessive, over the west-central plain.

Thereafter clear weather appeared throughout the several basins until Jan. 20, when extensive, though light, precipitation was recorded along and about the Ohio River and throughout parts of the watersheds of the Mississippi and Missouri Rivers as far north as Minneapolis, Minn., and Huron, S. D. On Jan. 21 a low barometer centered about Omaha, Neb., moving southwest into the upper part of the State of Mississippi on Jan. 22. On both days rainfall was general over the Ohio, the lower Mississippi and the Missouri, with 1.76 in. at Chicago; 1.2 at Davenport, Iowa; 1.43 at Keokuk; 1.33 at Peoria, Ill.; 1.08 at Springfield, Mo.; and 2.24 at Kansas City. A radical rise of the Mississippi followed from the run-off of the lower Missouri, the Illinois, Kaskaskia and smaller tributary rivers of the main stream, the St. Louis stage climbing from 6.8 ft. on Jan. 21 to 23.9 on Jan. 26.

Mild Disturbance in Upper Mississippi Area

An area of mild disturbance again appeared throughout the upper Mississippi and Missouri watersheds on Jan. 26, developing on Jan. 27 into a storm of marked intensity, centered near Davenport, Iowa. The severest precipitation occasioned by this disturbance was reported from Springfield, Mo., with 2.46 in.; Keokuk, Iowa, 1.30; Davenport, 1.14; and Madison, Wis., 1.10. Between Jan. 27 and Feb. 1 the Ohio, Mississippi and Missouri watersheds were generally involved, though precipitation was severe only at such localities as Madison, Wis., Davenport and Keokuk, Iowa, and Kansas City, Mo., at each of which the rainfall exceeded 1 in. on Jan. 27. At Springfield, Mo., on the same day 2.46 in. was recorded. On Jan. 29 Evansville, Ind., received 1.72 in.

Rain was general throughout the Ohio and middle Mississippi Valleys on Jan. 31, with such extremes as 2.78 in. at Cairo; 1.46 at St. Louis; 2.12 at Terre Haute, Ind.; 2.10 at Evansville; and in excess of 1 in. at Memphis, Toledo, Fort Wayne and Chicago. Light rains continued over the Ohio on Feb. 1, with a maximum of 1.16 in. at Nashville, Tenn. Since Feb. 1 the weather throughout all upper drainage areas has been cold and clear.

Flood Crest at Cairo Reaches 53.4 Ft.

The gage at Cairo had receded from 48.1 ft. on Jan. 21 to 47.1 on Jan. 24. On Jan. 22 the St. Louis gage stood at 7.7 ft., though by Jan. 24 it had jumped to 20.2 ft. The St. Louis gage thereupon proceeded through an essentially unbroken rise to 31.7 ft. on Jan. 31, simultaneously with a rise in the lower Ohio, the extent of which is suggested by a maximum gage at Evansville of 43.4 ft. on Jan. 17, which, receding to a minimum of 35.2 ft. on Jan. 26, thereafter rose more or less continuously to the crest of 40.2 ft. on Feb. 4, its flood stage being 35. The effect of the conditions outlined subsequent to Jan. 22, involving a conjunction of the already flooded Ohio with a rapidly rising Mississippi, has been a crest at Cairo today (Feb. 5) of 53.4 ft. This compares with a flood crest in 1913 of 54.7 ft.—the highest of record.

Summarizing, it may be stated that the present floods in the Ohio, and in the Mississippi from St. Louis to the White River, are mainly the effect of the unseasonable weather that existed over the several watersheds btween the middle of December, 1915, and the first of February, 1916, with the result of general and unusually

*Corps of Engineers, U. S. A., in charge of First and Second Mississippi River Districts, Memphis, Tenn.

persistent rainfall instead of the customary snow, complicated by severe, though not extreme or record, precipitation over the Ohio River basin on and about Dec. 17 and 18, Dec. 28 and 30, Jan. 12, 20, 21, 22 and 27 and over the upper Mississippi and lower Missouri basins on Jan. 21, 22, 27 and 31.

The junction at Cairo of the Mississippi and Ohio Rivers, both beyond their flood stages, has brought to the lower valley, as far as the White River, a flood plane that will probably be deficient of that of 1913 by about 1 ft. The anticipated maximum gage at Memphis will be between 45 and 46 ft., its flood stage being 35. The flood of 1913 was recorded at Memphis as 46.5 ft. At Helena a maximum gage of 53 to 54 ft. is probable, that of 1913 being 55.2. The Helena flood stage is 42 ft.

Effect of Floods on Levees and Levee Districts

The areas protected by Government levees between Cairo and the White River comprise what are known as the Upper St. Francis basin, in southeastern Missouri, with an area of 448,000 acres; the lower St. Francis basin of Missouri and Arkansas, with an area of 2,240,000 acres; the Reelfoot basin below Hickman, in Tennessee and Kentucky, of 200,400 acres; the Upper Yazoo basin, in the State of Mississippi, south of Memphis, with an area of 2,099,840 acres; and the White River basin, in Arkansas, below Helena, with an area of 582,400 acres. The total length of levees protecting these several districts is about 500 mi.

These levees are intact at date and give promise of successfully resisting the present water, except at the lower end of the White River system, in the vicinity of the Laconia Circle, where levee grades are seriously deficient. A large amount of levee work has been accomplished since 1913, and many miles of such weak and insufficient structures as then existed have since been strengthened and enlarged. Wherever the levee system has been brought to the height and section established by the Mississippi River Commission, no uneasiness is felt; and no emergency work has thus far been required beyond the unusual precaution of careful and systematic observation and patrol. A number of reaches exist throughout which the levees are far below standard requirements, notwithstanding which there is every reason to believe that they will pass the existing flood unbroken.

A private levee near Hickman, Ky., maintained by that town and the Mengel Box Co., was overtopped a few days since, submerging about 100 acres of town and factory property near the base of the Hickman Hills. This levee and a small corporation levee in Illinois, opposite Cape Girardeau, Mo., are the only ones known to have failed along the Mississippi north of the White River.

No engineering structures between Cairo and the White River have been destroyed or jeopardized, except the collapse of falsework and the consequent loss of some unassembled steel of the new bridge of the Arkansas & Memphis Railway Bridge and Terminal Co., under construction across the Mississippi at Memphis (see *Engineering News*, Dec. 23, 1915). This loss occurred at the relatively low stage of 26.6 ft., Memphis gage, owing to the accumulation above the falsework of river drift, which on a then rising river was moving downstream in such excessive quantities and under such adverse wind conditions that it could not be successfully shunted past the falsework or handled through it. The accumulation above the two spans involved, prior to collapse, is estimated to have amounted to two or more acres, intermatted to a depth of at least 20 ft.

High Flood Plane Below White River Indicated

The foregoing hasty outline of the causes and effects of the existing flood takes no consideration of conditions in the White and Arkansas Rivers nor of their consequences along the lower Mississippi to the Gulf. Both rivers are at record stages, incident to heavy precipitation over their runoff basins, commencing with the storm centering over Omaha on Jan. 21 and 22, followed by the storms of Jan. 27, 28, 29, 30 and 31. The unfortunate conjunction of the Arkansas and White, carrying record flood water, with the Mississippi at extreme, though not record, stage fixes the certainty of a flood plane for the lower Mississippi that will probably equal, and may exceed, the measurements of 1912 and 1913.

Reverting again to the Mississippi between Cairo and the White River, existing and prospective weather conditions justify the hope that the runoff of the Ohio and upper Mississippi watersheds is in the rivers, with no indication of additional atmospheric disturbance within such period as to further complicate or aggravate the situation. The rapid fall of the Mississippi between St. Louis and Cairo, now in progress, should insure a recession of the Cairo gage below the point of hazard. On the other hand, any immediate resumption of extreme, or even heavy, precipitation over extensive drainage areas of the upper valleys would bring into question a flood of ominous proportions.

United States Supreme Court on Special Assessments

The Supreme Court of the United States on Jan. 31 handed down a decision in a St. Louis suit to enforce payment of a special tax bill for street improvement. The court held that the form of assessment was unjust, as the property was not benefited.

The improvement in question was made under the old charter of the City of St. Louis, which required that two-fifths of the cost of the improvement should be assessed pro rata on the frontage and that three-fifths should be assessed on an area basis against all property within a line one-half way to the next public street.

This method of assessment was satisfactory where the streets were laid out at regular intervals; but in a number of instances, of which the street in question was one, there are large vacant tracts on one side of the improvement, and in some cases the next public street is nearly a mile away. This resulted in property a half mile from the improvement being assessed a part of the cost. The injustice of this was recognized, and in the new charter of the City of St. Louis, effective in August, 1914, the method of assesment was changed, one-third of the cost being assessed pro rata on a frontage basis and two-thirds on area basis against all property within the limits of a taxing district laid out by the Board of Public Service and established by ordinance passed by the Board of Aldermen. In other words, the Board of Public Service was given discretion to consider the benefit to any particular property.

It is seen therefore that the provisions in the new charter are directly in line with the decision of the Supreme Court and that if the taxing districts are laid out reasonably by the Board of Public Service, the decision of the court has no bearing on assessments made under the new charter. It has been estimated, however, that there are about a million dollars in tax bills still unpaid for work done under the old charter and that several hundred thousand dollars of these bills may be affected by the decision.

Upper Otay Concrete Arch Dam Stands Heavy Flood

As described in our last issue, the Upper Otay dam, near San Diego, Calif., a bold reinforced-concrete arch structure, was subjected to a severe test by the same flood that destroyed the Lower Otay rock-fill dam. A sheet of water 3 ft. deep passed over the crest of the

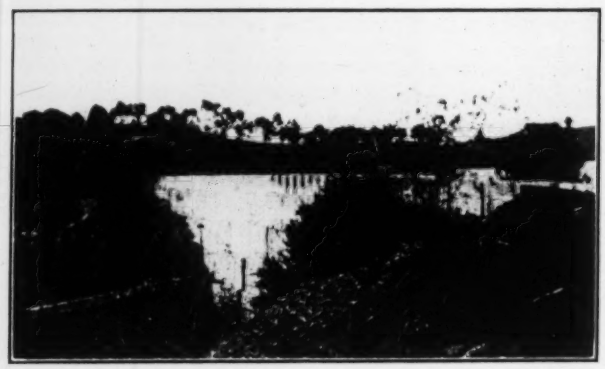


FIG. 1. DOWNSTREAM FACE OF UPPER OTAY DAM

Upper Otay dam while the flood was at its height, but it stood the test uninjured.

The occurrence is of especial interest, because the Upper Otay dam is one of the boldest arch dams ever constructed. Schuyler, in his treatise on reservoirs, refers to the Upper Otay dam as, with the exception of the famous Bear Valley dam, "the slenderest dam in California or any other part of the globe." Since the Upper Otay dam was built, however, there have been constructed in Australia, to form irrigation reservoirs, some arch dams that are even thinner than the Upper Otay dam. Information is not available as to whether these Australian dams have ever been subjected to the test of a full head of water behind them. The reservoir behind the Upper Otay dam was never filled before the recent flood.

FIG. 2. UPPER OTAY RESERVOIR NEARLY FULL. OCT. 27, 1915

The drawing herewith shows a plan, front elevation and cross-section of the Upper Otay dam and is reproduced from a detailed description of the dam published in *Engineering News* of Apr. 7, 1904. It will be seen that the thickness of the dam at the base, 75 ft. below the crest, is only 14 ft. The lowest 25 ft. of the dam, however, is very narrow, so that stresses due to arch action are not severe. As the view shows, the dam forms a very flat arch for so thin a structure, and by the time an elevation is reached where the arch becomes of considerable span the water pressure behind it is considerably reduced.

The accompanying views of the dam are reproduced from photographs taken by Allen Hazen, Consulting Engineer, on Oct. 27, 1915. As may be judged from these views, the water falling over the crest of the dam strikes directly upon the rocky sides of the cañon in which the dam is built. This rock is rather soft, and were the overfall to continue for a long time, serious erosion would probably occur. The flow over the crest due to the recent storm, however, was doubtless of short duration.

The design of the Upper Otay dam, as well as that of the Lower Otay dam, which was destroyed in the flood

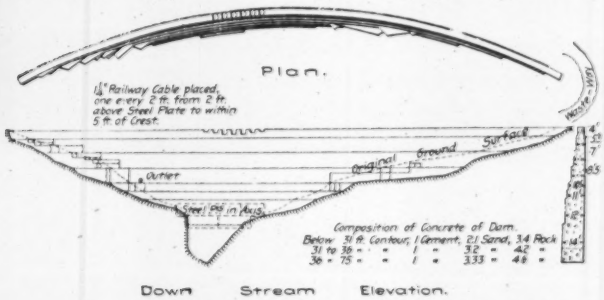

Plan.
½" Railway Cable placed, one every 2 ft. from 2 ft. above Steel Plate to within 5 ft. of Crest.
Surface
Original Ground
Outlet
Steel Pipe in Axis
Composition of Concrete of Dam.
Below 31 ft. Contour, 1 Cement, 2½ Sand, 3½ Rock
31 to 36 = " " 1 " 3½ " 4½ "
36 to 75 = " " 1 " 3⅓ " 4½ "
Down Stream Elevation.

FIG. 3. DETAILS OF UPPER OTAY CONCRETE-ARCH DAM NEAR SAN DIEGO, CALIF.

of Jan. 27, was due to E. S. Babcock, who at the time of their construction was president of the Southern California Water Co. Mr. Babcock long ago disposed of his interest in the company to John D. Spreckels, and the property was later transferred to the City of San Diego. Mr. Babcock, however, reserved a country home for himself close beside the Upper Otay dam. The disastrous failure of the lower structure must have been in plain sight from his house.

In commenting on the design of the Upper Otay dam, Schuyler, in his book on reservoirs, said: "The dam must be regarded as one of the most interesting of modern structures in that line, whose ultimate fate will always be looked upon with curiosity by the profession."

Judging by its behavior during the recent flood, the ultimate fate of the Upper Otay dam may be long delayed.

The Trans-African Railway from the Congo River to Dar-es-Salam, on the east coast, has been completed, the east link being the 200 mi. of line from Kabala on the upper Congo to Albertville on Lake Tanganyika. This link is in the Belgian colony of the Congo. The railway begins at the head of navigation on the Congo and extends 500 mi. to Lake Tanganyika, upon which there is steamer service. A railway built by the Germans extends from Ujiji, on the lake, to Dar-es-Salam, on the coast, about 800 mi.

Indiana Engineers Meet in Joint Convention

A joint meeting of the Indiana Engineering Society and the Indiana Sanitary and Water-Supply Association was held at the Claypool Hotel, Indianapolis, on Feb. 2 to 5. Matters relating to public affairs and policy predominated over those relating strictly to engineering.

Public utilities and their control received a good share of attention. The efficient operation of utility plants was dealt with by Charles Brossmann (Indianapolis), and in the discussion Mr. Ahara (Mishawaka) expressed the opinion that purchasing coal on the heat-unit basis is not advantageous under all conditions. Paul Hansen (Urbana, Ill.) suggested that small companies which cannot afford to pay salaries sufficient to command the services of the most efficient men can overcome this difficulty by arranging with consulting engineers for the periodical inspection of the plants with a view to proper maintenance and operation.

Municipal ownership was strongly advocated by F. F. Chandler (Indianapolis), who predicted its extension to water-supply, electric light, gas, ice making, garbage disposal, etc.

Mr. Garman, Chief Engineer of the Indiana Public Service Commission, discussed the difficulties of regulating public utilities by commissions. This system is now well established, but the paper presented some of the directions in which it needs to be strengthened. The author was very sure that municipal ownership will extend rapidly and widely.

A paper on "Standardization of Gravel for Concrete," by C. C. Brown (Indianapolis), brought up anew the question, "What is gravel?" Mr. Brown outlined investigations as to the characteristics of the various gravel deposits in Indiana and suggested that steps should be taken to prepare definite specifications for gravel to be used in making concrete.

One subject discussed, which, while old, is still of practical interest at this time, is the qualifications and compensation of the county surveyor. Under Indiana laws no technical qualification is required, but the surveyor may be in charge of extensive public engineering work. He may, however, make declaration that he is not an engineer and request the court to appoint a competent substitute. It has been urged that, as the law limits the pay of the surveyor to $4 a day, this must be also the limit for the substitute. The result would be that competent substitutes would not be obtainable. A committee of the society is to take up this matter with the state departments.

One evening was given up to a centennial meeting in recognition of the coming centennial celebration of the State of Indiana. Papers were presented reviewing 100 years of progress in public-health administration, by Doctor King, Assistant State Health Commissioner; in engineering, by Prof. W. K. Hatt, Purdue University; and in the field of water-supply, by W. G. Ulrich, Indianapolis Water Co.

The Indiana Sanitary and Water-Supply Association elected officers as follows: President, E. L. Loomis, Valparaiso; secretary, Dr. W. F. King, Indianapolis. The Indiana Engineering Society elected the following: President, Prof. L. W. Wallace, Indianapolis; secretary, Charles Brossmann, Indianapolis.

Arthur J. Baldwin President of the Hill Publishing Co.

Arthur J. Baldwin has been elected President of the Hill Publishing Co. to succeed the late John A. Hill. Mr. Baldwin is a prominent lawyer in New York City who has been for the past ten years Mr. Hill's personal attorney and business adviser and one of his most intimate friends. He is an executor of Mr. Hill's will and a trustee of his estate and has long been familiar with Mr. Hill's policies and plans.

Mr. Baldwin is a member of the law firm of Griggs, Baldwin & Baldwin. He is a graduate of Cornell University in the class of 1892. His law practice for the last ten years has been almost exclusively in connection with large business enterprises and particularly in connection with the legal end of engineering and contracting. He is at the present time treasurer of the Rogers Silver Plate

ARTHUR J. BALDWIN

Co.; treasurer of the Borough Development Co., which holds the contract for the removal of ashes from Brooklyn; treasurer of the Boston Development and Sanitary Co., which has a contract for garbage and ash disposal for the City of Boston; vice-president of the Automatic Fire Protection Co. and secretary of the Mississippi Wire Glass Co. He was a member of the New York Constitutional Convention of 1915.

Mr. Baldwin has arranged his business affairs so as to be able to take active charge as the head of the Hill company. He will continue unchanged the organization and policies established by the founder of the company.

Pennsylvania State Highway Work in 1915 was almost entirely that of maintenance. According to the records of the Maintenance Division, the improved roads maintained in 1915 to Oct. 1, are 8,262.68 mi. out of the total system of 10,200 mi. This total is subdivided as follows: Mileage of stone or other hard-surfaced roads, 1,880.48 mi.; mileage of flint, gravel and shale roads, 557.75 mi.; mileage of earth roads dragged and scraped, 5,824.45 mi. In addition the State Highway Department has resurfaced 396.99 mi. during the season to Oct. 1.